New capacity design methods for seismic design of ductile RC shear walls

Nouvelles méthodes de dimensionnement à la capacité pour la conception parasismique de murs ductiles en béton armé

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"The most incomprehensible thing about the universe is that it is comprehensible."
– Albert Einstein
RÉSUMÉ

Afin de réaliser des conceptions parasismiques économiques, les codes du bâtiment modernes permettent de réduire les forces sismiques si le système de résistance aux forces sismiques (SRFS) d’un bâtiment est conçu pour développer un mécanisme inélastique de réponse latérale. Le dimensionnement à la capacité (DAC) vise à assurer que le mécanisme inélastique se développe tel que prévu et qu’aucun mode de rupture non désiré ne survienne. Depuis l’édition 1984, cette approche de dimensionnement est recommandée dans la norme canadienne CSA A23.3 pour la conception parasismique de murs ductiles en béton armé dans le but d’offrir une résistance flexionnelle et en cisaillement suffisante pour confiner le mécanisme aux rotules plastiques identifiées et assurer une réponse latérale flexionnelle des murs. Pour un simple mur régulier, les exigences de DAC recommandées dans la norme supposent une déformation latérale du mur dans son premier mode de vibration latérale, et visent donc à contraindre le mécanisme inélastique à la rotule plastique prévue à la base du mur. On dit qu’on a une conception à rotule plastique unique (RPU). Malgré ces exigences, la norme CSA A23.3 ne recommandait pas, avant l’édition 2004, de méthodes pour déterminer des enveloppes de DAC pour le dimensionnement en flexion et en cisaillement de murs ductiles en béton armé sur toute leur hauteur. Seul le commentaire de la norme recommandait de telles méthodes. Toutefois, diverses études ont suggéré, principalement pour les murs simples, que l’application de ces méthodes pouvait conduire à des conceptions de murs vulnérables à la formation de rotules plastiques imprévues aux étages supérieurs et à une rupture potentielle par cisaillement, surtout à la base du mur, mettant en péril la réponse ductile flexionnelle du mur. Ces problèmes de conception résultent d’une sous-estimation de l’amplification dynamique causée par les modes latéraux de vibration ayant des fréquences supérieures à celle du mode fondamental. La norme CSA A23.3 2004 recommande maintenant des méthodes de DAC visant en partie à résoudre ces problèmes. Bien que ces méthodes n’aient pas encore été évaluées, leur formulation paraît déficiente dans la considération des effets d’amplification des modes supérieurs. Par conséquent, ce projet de recherche propose pour la norme CSA A23.3 de nouvelles méthodes de DAC considérant ces effets pour une conception à RPU de murs ductiles simples en béton armé utilisés comme SRFS de bâtiments à étages. Une évaluation de la performance sismique d’un système de murs ductiles réalisés dimensionné selon la norme CSA A23.3 2004 est d’abord réalisée afin d’évaluer les méthodes recommandées pour le DAC. Par la suite, une étude paramétrique basée sur des simulations dynamiques sophistiquées est menée afin d’identifier l’influence de divers paramètres sur les effets d’amplification des modes supérieurs, et donc sur la demande sismique en force, dans des murs ductile simples dimensionnés avec la norme CSA A23.3 2004. Enfin, une revue de diverses méthodes de DAC proposées dans la littérature et recommandées par des codes de conception pour une conception à RPU est réalisée. À partir des résultats de cette revue et de l’étude paramétrique, de nouvelles méthodes de DAC sont proposées, et une discussion sur les limitations de ces méthodes et sur leur applicabilité à divers systèmes de murs est présentée.

Mots-clés : Conception parasismique, murs ductiles en béton armé, méthodes de dimensionnement à la capacité, amplification dynamique des modes supérieurs
In order 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. The capacity design aims to ensure that the inelastic mechanism develops as intended and no undesirable failure modes occur. Since the 1984 edition, this design approach is implemented in the Canadian Standards Association (CSA) standard A23.3 for seismic design of ductile reinforced concrete (RC) shear walls with the objectives of providing sufficient flexural and shear strength to confine the mechanism to the identified plastic hinges and ensure a flexure-governed inelastic lateral response of the walls. For a single regular wall, the implemented capacity design requirements assume a lateral deformation of the wall in its fundamental lateral mode of vibration, and hence aim to constrain the inelastic mechanism at the expected base plastic hinge. This design is referred to as single plastic-hinge (SPH) design. Despite these requirements, CSA standard A23.3 did not prescribe, prior to the 2004 edition, any methods for determining capacity design envelopes for flexural and shear strength design of ductile RC shear walls over their height. Only its Commentary recommended such methods. However, various studies suggested, mainly for cantilever walls, that the application of these methods could result in multistorey wall designs experiencing the formation of unintended plastic hinges at the upper storeys and a high potential of undesirable shear failure, principally at the wall base, jeopardizing the intended ductile flexural response of the wall. These design issues result from an underestimation of dynamic amplification due to lateral modes of vibration higher than the fundamental lateral mode. The 2004 CSA standard A23.3 now prescribes capacity design methods intending in part to address these design issues. Although these methods have not been assessed yet, their formulation appears deficient in accounting for the higher mode amplification effects. In this regard, this research project proposes for CSA standard A23.3 new capacity design methods, considering these effects, for a SPH design of regular ductile RC cantilever walls used as SFRS for multistorey buildings. In order to achieve this objective, first a seismic performance assessment of a realistic ductile shear wall system designed according to the 2004 CSA standard A23.3 is carried out to assess the prescribed capacity design methods. Secondly, an extensive parametric study based on sophisticated inelastic dynamic simulations is conducted to investigate the influence of various parameters on the higher mode amplification effects, and hence on the seismic force demand, in regular ductile RC cantilever walls designed with the 2004 CSA standard A23.3. Thirdly, a review of various capacity design methods proposed in the current literature and recommended by design codes for a SPH design is performed. From the outcomes of this review and the parametric study, new capacity design methods are proposed and a discussion on the limitations of these methods and on their applicability to various wall systems is presented.

Keywords: Seismic design, ductile concrete cantilever walls, capacity design methods, higher mode amplification effects
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CHAPTER 1

Introduction

1.1 Context and problem description

Earthquakes are some of the most devastating natural disasters to affect mankind. Severe earthquakes can cause mass casualties resulting from structural damage in a short period without warning. In the last decade, earthquakes caused the deadliest disasters, almost 60% of all disaster-related mortality [United Nations International Strategy for Disaster Reduction, 2010], in part because they often occurred in populous urban areas with buildings designed with poor or without seismic structural standards and seismic force resisting systems (SFRSs). Earthquakes remain a serious threat as urbanization increases worldwide and many of the most populous cities in the world are near or on earthquake fault-lines. From a structural engineering point of view, this underlines the necessity of structural standards with adequate seismic provisions and efficient SFRSs to limit excessive structural damage under major earthquakes.

The efficiency of reinforced concrete (RC) shear wall systems for resisting lateral forces and controlling lateral drifts in multistorey buildings, particularly in tall ones, has long been recognized. In high seismically active regions, RC shear wall systems are the preferred SFRSs for all types of concrete building structures because they have shown during the major earthquakes of the last five decades to be the best systems for providing both life safety and property protection at the lowest cost, with a better performance with regard to life safety [Fintel, 1995; Kam and Pampanin, 2011; Lagos, 2011]. In fact, these systems have the capacity to withstand strong seismic shaking well beyond design level, though not without significant damages and economic losses. Typical shear walls for multistorey buildings are cantilever walls and walls coupled by coupling beams. Coupled walls are often used in residential and office buildings because they are more efficient and economical for lateral force resistance than cantilever walls simply interconnected by floor slabs.

In order to produce economical seismic designs, the modern building codes allow reducing seismic design forces if the SFRS of the 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 for flexural...
plastic behavior and all other regions of the structure and other possible behavior modes are provided with sufficient strength. This design approach, primarily developed in New Zealand [Park and Paulay, 1975], is referred to as capacity design and has been implemented in several codes around the world mainly because it could be applied in simple static linear analysis and design practices. Since the 1984 edition, this design philosophy is implemented in the Canadian Standards Association (CSA) standard A23.3 for seismic design of ductile RC shear walls. The implemented capacity design requirements apply essentially to wall structures that are substantially uniform and regular in strength and stiffness over the full height of the building. For such regular structures under lateral deformations, a plastic hinge is expected to form at the base of cantilever walls and at the coupling beam ends and the wall base of coupled walls, as illustrated in Figure 1.1. The capacity design requirements of the CSA standard A23.3 are based on these assumptions, and hence aim to constrain the plastic hinges at these locations. For a single wall, this results in a single plastic-hinge (SPH) design.

Despite capacity design requirements, the 1984 CSA standard A23.3 (A23.3-M84) [CSA, 1984] does not prescribe any capacity design methods to determine the flexural and shear strength of ductile RC walls over their height. It is the *Explanatory notes on CSA standard A23.3-M84* [CPCA, 1985] that specify these methods, which assume a first-mode response of the wall at its probable flexural capacity and hence a SPH formation at the wall base. The CPCA [1985] recommend a linear probable moment envelope for flexural
1.1. CONTEXT AND PROBLEM DESCRIPTION

strength design and, for shear strength design, a probable shear force envelope obtained by amplifying the design shear force diagram determined from the static procedure prescribed by the 1985 edition of the National Building Code of Canada [NRCC, 1985], as shown in Figure 1.2. The flexural and shear strengths of the wall must not be lower than these envelopes and a special ductile detailing must be provided over the hinging region \( (h_p) \).

![Figure 1.2 Capacity design methods recommended by CPCA [1985] for ductile RC walls.](image)

The 1994 edition of the CSA standard A23.3 (A23.3-94) [CSA, 1994] still does not prescribe any capacity design methods for flexural and shear strength design of ductile RC walls over their height, despite capacity design requirements. Once again, the *Explanatory notes on CSA standard A23.3-94* [CAC, 1995] specify these methods, which are the same as those recommended by the CPCA [1985], except for shear design, as shown in Figure 1.3. Actually CSA standard A23.3-94 requires accounting for dynamic amplification of shear forces when determining the wall shear strength. Because of this requirement, the CAC [1995] recommend amplifying the probable shear force envelope by a dynamic shear amplification factor \( (\beta_v \text{ in Fig. 1.3}) \), which is an adapted version of that specified in the New Zealand concrete design standard [NZS, 1995]. This requirement is the result of numerical studies [Filiatrault et al., 1992, 1994] on multistorey ductile shear wall buildings under design-level seismic motions showing a significant shear underestimation issue of the probable shear force envelope because of dynamic amplification effects, and hence a high potential of undesirable shear failure of ductile walls designed with this shear envelope. Such potential failure jeopardizes the intended ductile flexural wall response.
CHAPTER 1. INTRODUCTION

The dynamic amplification effects on the seismic force demand result from the significant contribution of 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 seismic force response of RC shear wall structures used as SFRS for multistorey buildings is generally dominated by the higher mode responses. The predominating contribution of higher lateral modes in the elastic response of such systems produces moment and shear force demand profiles over the wall height that are 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, as illustrated in Figure 1.4. These well-known effects are associated to elastic effects of higher lateral modes. An additional dynamic effect occurs when the system 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 satures and reduces with the first-mode period lengthening [Priestley, 2003; Sangarayakul and Warnitchai, 2004; Seneviratna and Krawinkler, 1994]. This dynamic amplification is associated to inelastic effects of higher lateral modes.

Recent Canadian numerical studies [Chaallal and Gauthier, 2000; Renaud, 2004; Tremblay et al., 2001] suggested that the capacity design methods recommend by the CAC [1995] (Fig. 1.3) can produce design strength envelopes that largely underestimate the seismic force demand on multistorey ductile RC walls under design-level ground motions because of an underestimation of higher mode amplification effects. This results in a possible
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From static code procedure
Actual profile

Flexural demand
Shear demand

Figure 1.4 Higher mode amplification effects on seismic force demand.

formation of unintended plastic hinges at the upper storeys and still a high potential of undesirable shear failure, specifically at the wall base. These observations are mainly for cantilever walls, though the same shear issue can occur for coupled walls whose shear strength design is based on the tension rather than the compression wall [Chaallal and Gauthier, 2000].

Despite the large improvements made to the 2005 edition of the NBCC [NRCC, 2005] and the 2004 edition of the CSA standard A23.3 (A23.3-04) [CSA, 2004], none of the new seismic design provisions of these Canadian codes specifically address the aforementioned likely design issues. Nevertheless, the new provisions provide a more rationale design approach for multistorey ductile RC shear wall buildings. Actually dynamic analysis is now the default method for seismic design, meaning that the elastic dynamic effects of higher modes are directly accounted for when the modal response spectrum method, the preferred NBCC dynamic analysis method, is used for seismic design. Moreover, CSA standard A23.3-04 now prescribes, for ductile RC shear walls, capacity design methods for determining capacity design envelopes for flexural and shear strength design of the wall regions above the base hinging region, as illustrated in Figure 1.5. In addition, to satisfy the shear strength requirements, the Explanatory notes on CSA standard A23.3-04 [CAC, 2006b] still recommend the same capacity design method based on the development of the probable flexural capacity at the wall base as that recommended by the CAC [1995]. However, neither the CSA standard A23.3-04 nor the CAC [2006b] recommend a method or a factor, as the $\beta_v$ factor recommended in CAC [1995] (see Fig. 1.3), to account for dynamic amplification of shear forces due to inelastic effects of higher modes, even though CSA standard A23.3-04 requires that this amplification be accounted for. Consequently, the very few published works [Boivin, 2006; Laporte, 2007] assessing the seismic perfor-
mance of multistorey ductile RC shear walls designed according to the 2005 NBCC and
the CSA standard A23.3-04 suggested that the wall shear demand can still be largely
underestimated, especially at the wall base, by the probable shear force envelope used
for capacity design. These works, however, did not really assess the new capacity design
methods prescribed by CSA standard A23.3-04 for flexural and shear strength design of
the wall regions above the base hinging region, and hence their adequacy is not known.
This needs to be addressed while considering the other capacity design requirements. So
far, only Boivin [2006] suggested that the new flexural method may produce conservative
capacity design moment envelopes for walls with large flexural overstrength at their base.
However, the new methods do not appear to be free from underestimation issues because
their formulation based on amplified elastic forces is tributary of the analysis method,
static or dynamic, used to derive these forces. The minor changes made to the seismic
provisions of the 2010 NBCC [NRCC, 2010] do not address any of the aforementioned
issues.

![Figure 1.5](image)

**Figure 1.5** Capacity design methods prescribed by CSA standard A23.3-04 and
recommended by CAC [2006b] for ductile RC walls.

In summary, the seismic provisions of the current edition (2004) of the CSA standard A23.3
for ductile RC shear walls can produce multistorey wall designs that, under design-level
seismic motions, may experience, with inappropriate detailing, the formation of unintended
plastic hinges at the upper storeys and present a high potential of undesirable shear failure,
mainly at the wall base, jeopardizing the intended ductile flexural response of the wall.
These design issues would result from a deficiency of the capacity design methods in
accounting for the higher mode amplification effects. Since ductile RC shear walls are
1.1 CONTEXT AND PROBLEM DESCRIPTION

the preferred SFRSs for multistorey buildings in the high seismic regions in Canada, particularly the West Coast (see Fig. 1.6), adequate capacity design methods for these walls are more than needed.

Figure 1.6 Historical earthquakes in or near Canada from 1627 to 2010
1.2 Objectives

From the problem description previously outlined, this research project proposes for CSA standard A23.3 new capacity design methods, considering higher mode amplification effects, for determining, for a SPH design, capacity design envelopes for flexural and shear strength design of regular ductile RC cantilever wall structures used as SFRS for multi-storey buildings. The research focusses on cantilever walls because higher mode amplification effects are usually much more important in cantilever walls than in coupled walls. From the general objective, it results the following specific objectives:

1. To assess the capacity design methods prescribed by the CSA standard A23.3-04 for flexural and shear strength design of ductile RC shear walls;

2. To investigate 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 walls;

3. To study the influence of various parameters on the higher mode amplification effects, and hence on the seismic force demand, in ductile cantilever walls;

4. To propose for CSA standard A23.3 new capacity design methods based on the main parameters affecting the seismic force demand on these walls.

1.3 Research significance

This research project is significant in three aspects: the regulatory, professional and research aspects. This research project proposes new capacity design methods to correct the deficiencies of the current methods of the CSA standard A23.3 for flexural and shear strength design of regular ductile RC cantilever walls. Actually the higher mode amplification effects in the inelastic regime are currently not taken into account. The proposed methods are the first Canadian methods to account for these effects and to be adapted to current Canadian codes. If these methods are implemented in the CSA standard A23.3, it is strongly believed that their application will produce safer and more predictable wall designs. This contribution would be major because the CSA standard A23.3 is the regulation for design of concrete structures in Canada.

Because of their parametric formulation, the proposed methods provide to engineers a better understanding of the parameters influencing the most the higher mode amplification effects in ductile RC cantilever walls, and hence enable them to make sound decisions when
designing such walls. This is significant because it gives to engineers a better control and therefore a greater level of confidence on their designs.

The last significant impact of this project is on the research in seismic engineering. Actually it is the first time that capacity design methods for RC walls are based on dynamic simulations accounting for the inelastic shear-flexure-axial force interaction according to the modified compression field theory (MCFT) and the disturbed stress field model [Vecchio, 2000]. The MCFT is the fundamental theory implemented in CSA standard A23.3 for shear strength design.

1.4 Document structure

In order to achieve the objectives of this research project, the work is broken down in three stages which are presented in three separate chapters of this document. Each chapter is actually a scientific journal paper.

In Chapter 2, the seismic performance of a 12-storey ductile RC core wall system designed according to the 2005 NBCC and the 2004 CSA standard A23.3 is assessed. This study assesses, from a realistic case under design-level ground motions and above, the adequacy of the capacity design methods prescribed by the CSA standard A23.3-04 for flexural and shear strength design of ductile RC shear walls. This study is an extension of the work of Boivin [2006] in which the same core wall building was studied. However, in Boivin [2006], the seismic design of the studied wall system was based on the capacity design methods recommended by the CAC [1995] rather than on those prescribed by the CSA standard A23.3-04 mainly because only draft versions of the CSA standard A23.3-04 with sometimes incomplete provisions were available at that time. In addition, the assessment of Boivin [2006] was only for design-level ground motions and did not include a damage assessment based on the continuum damage mechanics.

Chapter 3 presents first a short literature review on higher mode amplification effects in RC walls aiming to identify parameters that can have a significant influence on these effects. Afterwards, an extensive parametric study investigating the influence of various parameters on these effects, and hence on the seismic force demand, in regular ductile RC cantilever walls designed with the 2010 NBCC and the 2004 CSA standard A23.3 is performed. The objective of this study is to determine the parameters affecting the most the seismic force demand on these walls under design-level ground motions and to assess in a more general way the adequacy of the capacity design methods prescribed by the CSA standard A23.3-04 for flexural and shear strength design of such walls.
Chapter 4 presents first a review of 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 walls. This review aims to bring out the limitations of the current methods in estimating the seismic force demand on ductile walls whose seismic force response is governed by higher lateral mode responses. From the outcomes of this review and the parametric study conducted in Chapter 3, new capacity design methods are proposed for the CSA standard A23.3 for determining capacity design moment and shear envelopes for a SPH design of regular ductile RC cantilever walls considering higher mode amplification effects. Also a discussion on the limitations of the proposed methods and on their applicability to various wall systems is presented.

Finally, in Chapter 5, the main conclusions of the whole work are summarized and future research topics ensuing from this work are proposed.
CHAPTER 2

Seismic Performance of a Ductile Wall System

Foreword

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Titre français: Performance sismique d'un système de murs ductiles en béton armé de 12 étages dimensionné selon le Code national du bâtiment 2005 et la Norme A23.3 2004 de l'Association canadienne de normalisation

Paper's contribution to this project: This paper contributes to the thesis by showing from a realistic case study that the seismic provisions of the 2005 National building code of Canada (NBCC) and the 2004 Canadian Standards Association (CSA) standard A23.3 can produce capacity design shear envelopes that largely underestimate the seismic shear force demand on seismic force resisting systems made up solely of ductile concrete cantilever or coupled walls whose lateral response under design-level ground motions is dominated by lateral modes of vibrations higher than the fundamental lateral mode. This shows that the capacity design method prescribed by the 2004 CSA standard A23.3 for shear strength design can be deficient in estimating seismic shear force demand on such systems.
Abstract: This paper presents an assessment of the seismic performance of a ductile concrete core wall used as a seismic force resisting system for a 12-storey concrete office building in Montréal, designed according to the 2005 National building code of Canada (NBCC) and the 2004 Canadian Standards Association standard A23.3. The core wall consists of a cantilever wall system in one direction and a coupled wall system in the orthogonal direction. The building is analyzed in the nonlinear regime. The main conclusion from this work is that the capacity design shear envelope for the studied wall structure largely underestimates that predicted, primarily in the cantilever wall direction, and this in turn significantly increases the risk of shear failure. This issue is essentially due to (i) an underestimation by the new NBCC spectral response acceleration of the higher mode responses of a reinforced concrete wall structure whose seismic response is dominated by higher modes; and (ii) a deficiency in the capacity design method in estimating the wall shear demand on such walls, even when their behavior is lightly inelastic.

Key words: 2005 NBCC, 2004 CSA standard A23.3, ductile concrete cantilever and coupled shear wall systems, seismic design, higher mode effects, seismic shear demand.

Résumé: Cet article présente une évaluation de la performance sismique d'un système de murs de contreventement ductiles utilisé comme système de résistance aux forces sismiques pour un bâtiment de 12 étages en béton armé dimensionné selon le CNBC 2005 et la Norme CSA A23.3 2004, et situé à Montréal. Le système de murs se comporte comme un mur en cantilever dans une direction et comme un mur couplé dans la direction orthogonale. Le bâtiment est analysé dans le régime non-linéaire. La conclusion principale de ce travail est que l'enveloppe de dimensionnement à la capacité en cisaillement sous-estime largement celle prédite, surtout dans la direction du mur en cantilever, et que ceci en retour augmente significativement le risque de rupture par cisaillement. Ce problème est essentiellement causé par (i) une sous-estimation par l'accélération spectrale du CNBC des réponses des modes supérieurs de vibration d'un mur en béton armé dont la réponse sismique est dominée par les modes supérieurs; et (ii) une déficience de la méthode de dimensionnement à la capacité à estimer la demande en cisaillement sur de tels murs, même lorsque leur comportement est légèrement inélastique.

2.1. Introduction

Recent Canadian numerical studies [Renaud, 2004; Tremblay et al., 2001] suggested that the seismic design strength envelopes for ductile reinforced concrete (RC) shear walls determined in accordance with the requirements of the 1995 edition of the National Building Code of Canada [NRCC, 1995] and the capacity design considerations specified in the 1994 edition of Canadian Standards Association (CSA) standard A23.3 for the design of concrete structures (A23.3-94) [CSA, 1994] may underestimate the seismic shear and flexural demands on cantilever walls subjected to design-level ground motions. Comparable observations were also reported by Amaris [2002] for similar capacity design considerations, even for ground motion intensities lower than that of the design level. For ductile RC coupled wall systems designed according to the 1995 NBCC and CSA standard A23.3-94, no such issues have been reported in the published literature, except for systems whose shear strength design is based on the tension wall rather than the compression wall [Chaallal and Gauthier, 2000] and for systems subjected to seismic events much more severe than that used for design [Renaud, 2004; White and Ventura, 2004]. The underestimation issue of the capacity-based seismic design envelopes for flexural and shear strength designs would primarily result from a large underestimation of the dynamic magnification effects due to lateral modes of vibration higher than the fundamental lateral mode.

Although the improvements made to the seismic design provisions of the 2005 edition of the NBCC [NRCC, 2005] and the 2004 edition of the CSA standard A23.3 (A23.3-04) [CSA, 2004] do not specifically address the aforementioned issue, they provide a more rational seismic design approach for ductile RC shear wall systems. Very few published works [Panneton et al., 2006] have studied the seismic performance of such systems resulting from the application of these new design codes. In this regard, this paper presents an assessment of the seismic performance of a ductile RC shear wall system used as a seismic force resisting system (SFRS) for a 12-storey RC building designed and detailed according to the 2005 NBCC and CSA standard A23.3-04.

In this work, some of the new seismic design provisions of the 2005 NBCC and CSA standard A23.3-04 are examined, emphasizing their application for ductile concrete shear wall structures. Insight is also given into how higher mode effects on shear forces in such structures are addressed in these codes and how the capacity design strength envelopes for these structures are determined. The seismic design of the wall system is briefly presented as are the inelastic structural models, analysis parameters, and earthquake inputs used for the seismic performance assessment.
2.2 Canadian seismic design code provisions

2.2.1 2005 National building code of Canada

In contrast with previous editions of the NBCC, dynamic analysis is now the default analysis method for seismic design. The traditional equivalent static force procedure can still be used, but only for specific cases. Independently of the design procedure used, a minimum base shear force, \( V \), must be considered in determining the design shear force at the base of the building:

\[
V = \frac{S(T_a) M_v I_E W}{R_d R_o}
\]  

where \( S(T_a) \) is the new design spectral response acceleration, expressed as a ratio of gravitational acceleration, for the fundamental lateral period of vibration, \( T_a \), of the building in the loading direction of interest; \( M_v \) is a new factor to account for higher mode effects on base shear; \( I_E \) is the earthquake importance factor of the building \((0.8 \leq I_E \leq 1.5)\); \( W \) is the seismic weight of the building; \( R_d \) is the ductility-related force modification factor \((1.0 \leq R_d \leq 5.0)\); and \( R_o \) is a new overstrength-related force modification factor \((1.0 \leq R_o \leq 1.7)\) that accounts for the dependable portion of reserve strength in the SFRS.

The design spectral acceleration, \( S(T) \), is determined as:

\[
S(T) = F_a S_a(T) \quad \text{or} \quad F_v S_a(T)
\]  

where \( S_a(T) \) is the 5\% damped spectral response acceleration at period \( T \) determined for a probability of exceedance of 2\% in 50 years at a median confidence level; and \( F_a \) and \( F_v \) are new acceleration- and velocity-based site coefficients, respectively. Both site coefficients represent the amplification of seismic motions due to ground conditions. The \( S_a(T) \) is a uniform hazard spectrum (UHS) where the spectral accelerations at different periods are calculated at the same probability of exceedance for a specific geographical location.

The \( S_a(T) \) values given in the 2005 NBCC were determined for the reference ground condition of very dense soil or soft rock. The fundamental lateral period of vibration, \( T_a \), of the building can be determined either with the NBCC empirical relation for the SFRS of interest or from established methods of mechanics using a structural model that complies with NBCC requirements. For shear wall structures, \( T_a \) determined from the latter methods cannot be taken greater than 2.0 times that determined with the empirical relation, which is now \( 0.05(h_n)^{3/4} \) where \( h_n \) is the building height above the base in meters, and \( n \) is the number of storeys. This limitation is to ensure that computed \( T_a \) values will
2.2. CANADIAN SEISMIC DESIGN CODE PROVISIONS

not be much greater than those measured in actual buildings, as structural models tend to be more flexible than the actual building.

The new $M_v$ factor in Eq. (2.1) accounts for the dynamic magnification of base shear due to higher modes. The derivation of this factor can be found in Humar and Mahgoub [2003]. The $M_v$ values specified in the 2005 NBCC are a function of the type of SFRS, $T_a$, and the shape of the spectral response acceleration, $S_a(T)$. Table 2.1 gives the $M_v$ values specified in the 2005 NBCC for shear walls. A ratio $S_a(0.2)/S_a(2.0) < 8.0$ is typical for the western Canadian regions, and a ratio $S_a(0.2)/S_a(2.0) \geq 8.0$ is typical for the eastern Canadian regions. For $T_a > 1.0$, Table 2.1 indicates that the dynamic magnification of base shear due to higher modes is more significant for eastern regions than for western regions and increases with increasing $T_a$.

Table 2.1 Values specified in the 2005 NBCC for shear walls of the factor $M_v$ to account for higher mode effects on base shear.

<table>
<thead>
<tr>
<th>$S_a(0.2)/S_a(2.0)$</th>
<th>Type of SFRS</th>
<th>$T_a \leq 1.0$</th>
<th>$T_a \geq 2.0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 8.0</td>
<td>Coupled wall*</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Shear wall</td>
<td>1.0</td>
<td>1.2</td>
</tr>
<tr>
<td>≥ 8.0</td>
<td>Coupled wall</td>
<td>1.0</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>Shear wall</td>
<td>1.0</td>
<td>2.5</td>
</tr>
</tbody>
</table>

* Coupled wall is a shear wall system with coupling beams in which at least 66% of the base overturning moment resisted by the entire wall system is carried by the earthquake-induced axial forces in walls resulting from shear in the coupling beams.

It is important to note that, although both factors were determined to amplify the shear forces produced by the common code-specified static lateral force distribution, the $M_v$ factor is not equivalent to the dynamic shear magnification factor, $\omega_v$, specified in the New Zealand concrete design standard [NZS, 1995] for the shear strength design of ductile concrete shear wall structures. The $M_v$ factor only takes into account the magnification of base shear due to elastic effects of higher modes, whereas $\omega_v$ accounts for the magnification of shear forces due to inelastic effects of higher modes [Blakeley et al., 1975]. In addition, it is of interest to note that, unlike $M_v$ values, $\omega_v$ values depend only on the fundamental lateral period of vibration (number of storeys) of the building: $\omega_v = 0.9 + n/10$ for buildings up to six storeys and $1.3 + n/30$, to a maximum of 1.8, for taller buildings, where $n$ is the number of storeys. As noted by Priestley [2003], the current form of $\omega_v$ is deficient in capturing all significant causative parameters. Consequently, various numerical works [Amaris, 2002; Bachmann and Linde, 1995; Panneton et al., 2006] suggest that the
actual inelastic dynamic shear magnification in cantilever walls can be much larger than that predicted by \( \omega_v \). However, experimental investigations on this high magnification issue are needed, since the limited experimental work [Eberhard and Sozen, 1993] published so far does not report such large shear magnifications.

### 2.2.2 CSA standard A23.3-04

As linear analysis is generally used to predict earthquake design actions, the seismic design provisions of CSA standard A23.3-04 for ductile RC walls are based on capacity design principles. These provisions are mainly for wall structures that are substantially uniform and regular in strength and stiffness over the full height of the building.

**Flexural strength design**

For ductile RC walls designed for a single plastic hinge at the wall base, CSA standard A23.3-04 provides new capacity design provisions to prevent unexpected flexural yielding above the assumed base hinging region, which is taken to be at least 1.5 times the wall length, \( \ell_w \), in the direction under consideration (Fig. 2.1). From these provisions, a capacity moment envelope can be determined, corresponding to the development of the factored moment resistance \( (M_r) \) of the wall section over the assumed base hinging region. Note that \( M_r \) is calculated with factored material strengths, which are lower than their specified values. The capacity moment envelope is obtained by amplifying the NBCC design (factored) overturning moments for the wall, obtained from linear analysis, by the ratio of \( M_r \) to the factored moment, \( M_f \), both calculated at the top of the assumed plastic hinge region, as shown in Fig. 2.1. The factored moment resistance of the wall above the assumed hinging region must be set to match or exceed the resulting capacity moment envelope.

Previously, the Explanatory notes on CSA standard A23.3-94 [CAC, 1995] suggested that the capacity moment envelope be taken as a probable moment envelope varying linearly from the top of the assumed hinging region to the top of the wall, as illustrated in Fig. 2.1. This envelope assumes a first-mode lateral behavior of walls after plastic hinge formation at the wall base. The probable moment resistance, \( M_p \), of the wall above the hinging region had to match or exceed this linear envelope. However, recent numerical works [Renaud, 2004; Tremblay *et al.*, 2001] suggested that this approach might not prevent the formation of unintended plastic hinges above the base of multistorey walls due to higher mode effects. Note that \( M_p \) is calculated with a concrete compressive strength \( (f'_c) \) at its specified value and an equivalent steel yield stress of 1.25 times its specified value \( (f_y) \).
2.2. CANADIAN SEISMIC DESIGN CODE PROVISIONS

Figure 2.1 Capacity design moment envelopes for ductile RC walls.

Shear strength design

To satisfy the shear strength requirements of CSA standard A23.3-04, the factored shear resistance, $V_r$, of a ductile RC shear wall must not be less than the shear corresponding to the development of the probable moment capacity of the wall at its plastic hinge locations, accounting for the magnification of shear forces due to inelastic effects of higher modes. CSA standard A23.3-04 limits this probable envelope to a shear resulting from the NBCC design load combinations, which include earthquake with load effects calculated using $R_dR_o = 1.0$. It is of interest to note that CSA standard A23.3-04 does not prescribe any method to determine the probable shear envelope, nor any method to account for the inelastic dynamic shear magnification effect. For a single-base-hinge design of walls, the Explanatory notes on the CSA standard A23.3-04 [CAC, 2006b] suggest, as in the previous edition, that the probable shear envelope, $V_p$, on walls be estimated by amplifying the NBCC design shear force envelope, $V_f$, for the wall, obtained from linear analysis, as follows:

$$V_p = \gamma_p V_f = \left( \frac{M_p}{M_f} \right)_{base} V_f$$  \hspace{1cm} (2.3)

where $\gamma_p$ is the probable wall overstrength factor, and the ratio $M_p/M_f$ is calculated at the wall base. CAC [2006b], however, indicates that no Canadian method is available at this time to account for the magnification of shear forces due to inelastic effects of higher modes. Even the adaptation of the New Zealand’s dynamic shear magnification factor $\omega_v$ to Canadian codes suggested in CAC [1995] is no longer considered in the new edition [CAC, 2006b]. This appears to be a major issue because recent studies [Panneton
et al., 2006; Renaud, 2004; Tremblay et al., 2001] suggest that $V_p$ (Eq. (2.3)) considerably underestimates the shear demand on ductile RC walls subjected to design-level ground motions.

For a single-base-hinge design of ductile walls, CSA standard A23.3-04 now provides an additional requirement for the shear strength design at all elevations above the assumed plastic hinge height of $1.5\ell_w$ (Fig. 2.1). At each section above this height, the factored shear forces, $V_f$, must be scaled up by the same ratio $M_r/M_f$ used to determine the capacity moment envelope above the hinging region. $V_r$ of the wall must not be less than the maximum of the resulting shear envelope and the probable shear envelope, $V_p$, above the hinging region.

2.3 Studied building

2.3.1 Description

This research project studied a 12-storey RC office building located in Montréal and founded on soft rock, which is the reference ground condition (Class C) in the 2005 NBCC. The structural configuration and dimensions of the building structure are shown in Fig. 2.2. The structure is made of normal-density concrete and steel reinforcement with specified strengths $f'_c = 30$ MPa and $f_y = 400$ MPa. The SFRS is a central elevator core wall bracing a flat slab-column system with spandrel beams located along the exterior edges of each floor. The core wall extends one storey above the roof of the building, forming an elevator penthouse on the 13th floor. The core wall cross section measures $6 \text{ m} \times 8 \text{ m}$, center to center, and its thickness is 400 mm, for stability considerations, over the entire height of the wall. The core wall is composed of two C-shaped walls connected at the level of each floor by two $1 \text{ m}$ deep coupling beams with a span-to-depth ratio of 1.8. This configuration results in a coupled wall system in the east-west (E-W) direction and a cantilever wall system in the north-south (N-S) direction. It is noted that the building is very similar to the sample building in the 2006 edition of the Canadian Concrete Design Handbook [CAC, 2006a]. Herein, however, coupling beams are less slender, resulting in a more heavily coupled wall system. The dimensions of the other structural components of the building and the complete design and detailing of the building structure can be found in Boivin [2006]. Only a brief overview of the seismic design of the core wall is presented in the following sections.
Figure 2.2 Reinforced concrete (RC) core wall building.
2.3.2 Seismic design according to the 2005 NBCC

The core wall was designed to resist 100% of the earthquake loads and their effects, as required by the 2005 NBCC. An $R_d = 3.5$ was used for the cantilever wall direction and 4.0 for the coupled wall direction, as the coupled wall system can be considered fully coupled (degree of coupling greater than 66%).

The NBCC earthquake design loads were determined from a linear modal response spectrum analysis using the design acceleration response spectrum, $S(T)$, for Montréal with site coefficients $F_a = F_v = 1.0$ (soft rock). The dynamic analysis showed that the building structure was torsionally sensitive, and hence irregular, as defined by the 2005 NBCC. Consequently, the code-specified static procedure was not permitted for seismic design. The total lateral responses in each principal direction were obtained by combining the spectral responses of the first three lateral modes with the SRSS (square root of the sum of the squares) method.

Table 2.2 gives the parameters used to calculate $V$, $V_{dy}$ (which is the base shear obtained from linear dynamic analysis) and the resulting NBCC design base loads (bold values) for the building in each principal direction. The NBCC design base shears are equal to the larger of $V$ and $V_{dy}$, as the building structure is irregular in torsion. As permitted, the NBCC earthquake design loads are based on $T_a$ computed from modal analysis. The $T_a$ values are 1.74 s for the cantilever wall direction and 1.41 s for the coupled wall direction. These values do not exceed the NBCC-specified limit of two times the $T_a$ value calculated with the NBCC empirical relation, which gives $T_a = 0.87$ s. The use of the computed $T_a$ values rather than the empirical one has significantly reduced the earthquake design loads, as indicated in Table 2.2. This is due, in part, to the design spectrum shape for Montréal where the $S(T_a)$ values are reduced by about 50% and 60% as the period values increase from 0.87 s to 1.41 s and 1.74 s, respectively.

The estimated overall drifts and interstorey drifts of the building structure at design displacements, including the inelastic part, are low. In each principal direction, the overall building drift and the maximum interstorey drift for all storeys are not greater than 0.30%, which is significantly less than the NBCC limit of 2.5% for this building.

Figure 2.3 illustrates the contribution of higher lateral modes on the NBCC earthquake design force profiles over the entire height of the building. It indicates that higher mode effects play a major role in the seismic forces applied to the studied building structure, especially in the north-south direction.
Table 2.2 NBCC 2005 earthquake design loads for the studied building.

| Seismic loading direction | Type of SFRS | $T_a$ (s) | $S(T_a)$ (g) | $M_e$ | $R_d$ | $R_o$ | $I_E$ | $W$ (kN) | Base shear $V$ (kN) | $V_{dyn}$ (kN) | $V_{do}$ (kN) | $M_{do}$ (kN-m) | $T_{do}$ (kN-m) | Force reduction (%) |
|--------------------------|--------------|-----------|---------------|------|------|------|------|--------|-------------------|---------------|---------------|----------------|----------------|------------------|------------------|
| E-W                      | DFCW         | 0.87      | 0.19          | 4.0  | 1.7  | 1.0  | 0.87 | 86626  | 2452             | 1760          | 2452          | 44060          | 7455            | Ref.            |
| E-W                      | DFCW         | 1.41      | 0.10          | 4.0  | 1.7  | 1.0  | 0.87 | 86626  | 1355             | 1760          | 1760          | 32017          | 5351            | -28.2           |
| N-S                      | DCW          | 0.87      | 0.19          | 3.5  | 1.6  | 1.0  | 0.87 | 86626  | 2978             | 2062          | 2978          | 41958          | 9053            | Ref.            |
| N-S                      | DCW          | 1.74      | 0.07          | 3.5  | 1.6  | 1.0  | 1.74 | 86626  | 1938             | 2062          | 2062          | 29057          | 6269            | -30.8           |

Note: Force values in bold are those used for design. $V_{dyn}$, factored base shear obtained from SRSS; $V_{do}$, $M_{do}$ and $T_{do}$, shear force, overturning moment, and (accidental) torsional moment, respectively. The NBCC base overturning modification factor $J$ is 0.93 (0.87 s) and 0.82 (1.41 s) for DFCW and 0.85 (0.87 s) and 0.51 (1.74 s) for DCW.

- DCW, ductile cantilever wall; DFCW, ductile fully coupled wall.
- Period calculated with the NBCC empirical relation.
- Period computed from modal analysis.
Figure 2.3 NBCC seismic design force profiles over building height in both directions: (a) shear force; (b) overturning moment.
2.3. STUDIED BUILDING

2.3.3 Seismic design according to CSA standard A23.3-04

Because of the uniform structural configuration of the core wall, a single-base-hinge design is adopted for the walls. In the coupled wall direction, the coupling beams are designed to yield prior to the walls under a pushover loading. The resulting detailing characteristics of the core wall are as follows:

1. The lower first three storeys of the walls are detailed as a plastic hinge region. This height is governed by the wall length \( \ell_w \) in the north-south direction and is higher than the required minimum height of \( 1.5\ell_w \).
2. The flexural (vertical) reinforcement in the walls is governed by the required minimum reinforcement over the entire height of the core wall.
3. The required minimum flexural reinforcement in the assumed plastic hinge region \((1.5\ell_w)\) is extended along the height of the wall up to storey 9, inclusive. Above this storey, there is a curtailment of the flexural reinforcement. This curtailment is based on the factored moment resistance \((M_r)\) of a C-shaped wall matching or exceeding the capacity design moment envelope prescribed by CSA standard A23.3-04, as illustrated in Fig. 2.4a.
4. The shear (horizontal) reinforcement in the assumed base hinging region \((1.5\ell_w)\) is governed by the shear strength required to develop the probable flexural capacity \((V_p, Eq. (2.3))\) of a C-shaped wall in the north-south direction. The plastic hinge detailing for shear is extended up to storey 5, inclusive, to satisfy the shear strength requirement above the assumed hinging region. Above storey 5, the shear reinforcement in the walls is governed by the required minimum reinforcement, as shown in Fig. 2.4b.
5. Diagonal reinforcement is provided in the coupling beams. The beam reinforcement yielding strength matches as closely as possible the design beam shear envelope after up to 20% vertical redistribution was applied between beams, as suggested in CAC [2006b].

As shown in Fig. 2.4a for the north-south direction, the core wall has substantial flexural overstrength compared to the NBCC design envelope determined with either permitted values of \( T_a \). The wall overstrength factor \( \gamma_w \), which is defined as the ratio of nominal moment resistance \( M_n \) to the factored moment \( M_f \) at the base of the wall system, is about 3.6 for each principal direction. Note that \( \gamma_w \) would be about 2.5 if \( M_f \) is based on the empirical \( T_a \) value of 0.87 s. This large overstrength is due to the excess strength arising from the required minimum reinforcement. This is typical for core walls located...
Figure 2.4 Capacity design of a C-shaped wall in the north-south direction according to CSA standard A23.3-04: (a) in flexure; (b) in shear.
in moderate seismic regions such as Montreal but may not be the case in high seismic regions such as Vancouver, particularly for tall buildings for which the minimum base shear cutoff at a period of 2s usually applies. Despite the large flexural overstrength of the walls, the requirement for the sliding shear resistance at the wall base is barely satisfied, assuming that the construction joint is intentionally roughened. Note that the CSA standard A23.3-04 does not specify any upper limit for $\gamma_w$, only a lower limit of 1.3.

As shown in Fig. 2.4, for the north-south direction, the substantial flexural overstrength of the core wall at the base produces a significantly large design shear envelope $V_p$ for the walls, as compared with $V_f$. In the north-south direction, $V_p$ at the wall base is approximately equal to the maximum factored shear resistance ($V_{r,\max}$) allowed by CSA standard A23.3-04 for the base plastic hinge region.

As a result of the large flexural overstrength of the wall and the very low design overall drifts of the building, the calculated inelastic rotational demands, $\theta_{id}$, on the structural components of each wall system are very low, particularly for the walls, compared with the inelastic rotational capacities, $\theta_{ic}$, specified by CSA standard A23.3-04 for these components. Although the ductility requirement is satisfied ($\theta_{id} < \theta_{ic}$), the anticipated inelastic deformation level of the core wall is not in line with that assumed for design ($R_d > 3.0$).

The previous design issues indicate that excessive flexural overstrength can inhibit the intended large inelastic deformation of ductile RC walls under strong ground motions and hence the intended earthquake load reduction on the structure.

2.4 Modeling for inelastic analysis

2.4.1 Inelastic structural models

The seismic performance of the core wall was assessed from two-dimensional (2D) inelastic static (pushover) and time-history dynamic analyses using two finite-element structural analysis programs, namely RUAUMOKO [Carr, 2002] and EFICoS [Légeron et al., 2005]. RUAUMOKO mainly uses lumped plasticity beam elements to represent RC members, and EFICoS uses layered beam elements with uniaxial constitutive laws based on the continuum damage mechanics for concrete and the plasticity theory for steel. EFICoS was used primarily to validate RUAUMOKO models [Boivin, 2006].

The wall system in each principal direction was modeled as an isolated wall fully fixed at its base. This results in a cantilever wall model for the north-south direction and a coupled
Figure 2.5 RUAUMOKO structural models for inelastic seismic analysis. ($M_y$, yield moment; $M_{cr}$, cracking moment; $\varphi$, curvature)
wall model for the east-west direction. The walls and coupling beams were modeled with beam-line finite elements, which are located at member centroids. The end regions of the coupling beams were represented with rigid end extensions to account for the finite widths of the adjoining walls. Figure 2.5 shows the 2D RUAUMOKO wall models developed for analysis and the modeling parameters adopted to simulate the inelastic behavior in flexure of the structural members. Shear deformation was assumed to be linearly elastic, given the use of capacity design principles for shear strength design. The elastic shear stiffness of members was based on their effective shear area. As members were modeled with lumped plasticity elements, the modified bilinear Takeda hysteresis rule was used for coupling beams and the trilinear SINA hysteresis rule was used for walls. The latter rule makes it possible to capture the uncracked elastic response, which is significant for the wall under study, and to account for the effect of the tension stiffening of the concrete on the elastic response. The influence of tension stiffening on the dynamic responses of the wall models was investigated [Boivin, 2006]. It was observed, primarily for a lumped plasticity modeling, that not taking into account tension stiffening can lead to considerable underestimation of the wall shear demand when the wall structure behavior in flexure is lightly inelastic. The concrete tension-stiffening effect then was accounted for through the trilinear moment-curvature relationships determined for each storey from the sectional analysis program MNPhi [Paultre, 2001]. The strain hardening of steel was also taken into account. An elasto-perfectly plastic relationship was used as primary curve for the modified Takeda rule. Strength decay was not accounted for in the structural models. Note that the bending yield strength, \( M_y \), used for the hysteresis rules is the code-specified flexural strength of the member section. It was determined for a factored, nominal and probable resistance, as defined in CSA standard A23.3-04.

The wall models assumed that the building floors act as rigid diaphragms. Consequently, the mass of each storey was lumped at each floor level of the wall models. The total mass of the penthouse was lumped at the building roof. The seismic weight of the building used for analysis corresponded to the 2005 NBCC seismic loading case with 100% of dead loads, 50% of live loads, and 25% of snow load.

As shown in Fig. 2.5, a 2D inelastic structural model of the entire building structure in the north-south direction was also considered for analysis in order to assess the influence on the wall system of the added stiffness from structural components not part of the SFRS. A previous study [Renaud, 2004] on a similar core wall building suggested that this influence is negligible in the coupled wall direction because of the larger lateral stiffness of the SFRS, induced by the large coupling action, over the entire height of the building.
2.4.2 Earthquake ground motion histories

A suite of two historical and six simulated ground motion time histories was used for the inelastic time-history dynamic analysis. Table 2.3 gives the characteristics of these time histories and Fig. 2.6 illustrates their time history. Two excitation levels were considered: one corresponding to the design (median 2% in 50 year) UHS for Montréal, and another corresponding to the 84th percentile 2% in 50 years UHS for Montréal. The simulated time histories are design UHS-compatible accelerograms (Fig. 2.7a), as required by the 2005 NBCC for seismic design, and are representative of ground motions for magnitude-distance scenarios that dominate the seismic hazard of Montréal for the design probability level. The historical records were scaled in the frequency domain with the program SYNT [Naumoski, 2001] through an iterative suppression-raising technique to match the design and the 84th percentile 2% in 50 year UHS for Montréal over the entire period range of interest, as shown in Fig. 2.7b. Although spectrum-compatible accelerograms are unrealistic and physically inconsistent, various works [Léger et al., 1993] suggest that the use of such accelerograms for performance assessment purposes should not affect the validity of results as long as more than one accelerogram is used.

Note that the original 1940 El Centro record was also used as input motion. The 5% damped spectral acceleration response of this record has the particularity of fairly matching the 84th percentile UHS for Montréal over the long-period range (> 0.5s), as shown in Fig. 2.7b, but being lower over the short-period range. This means that, theoretically, the higher mode contribution under this record should be significantly less than those under a record matching the entire 84th percentile UHS.

2.4.3 Damping model

Damping is one of the main unknowns in dynamic analysis. Both the damping model and the damping level used for analysis have a strong influence on predictions. Consequently, two different viscous damping models were used for the inelastic time-history analysis, namely the constant (CD) and the Rayleigh damping (RD) models, both based on the initial elastic stiffness of the analyzed structural model. Two damping levels were considered for the constant damping model, namely 1% and 2% of critical. These values bound the range of typical modal damping values measured in actual undamaged mid-rise RC wall buildings [Boroschek and Yáñez, 2000]. The resulting CD models are referred to as the 1% CD and 2% CD models. For the Rayleigh damping model, referred to as the 2% RD model, a modal damping ratio of 2% of critical was used for the 1st and 12th lateral
2.4. MODELING FOR INELASTIC ANALYSIS

Table 2.3  Characteristics of the 2% in 50 years UHS-compatible simulated and historical ground motions for Montréal.

<table>
<thead>
<tr>
<th>Earthquake time history</th>
<th>UHS Compatibility</th>
<th>Trial No.</th>
<th>Scaling factor</th>
<th>PGA (g)</th>
<th>t_D (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simulated record</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$M$ 6.0 events at $R = 30$ km for eastern Canadian sites [Atkinson, 1999]</td>
<td>Scaled in time domain to match design (median) UHS</td>
<td>1</td>
<td>0.85</td>
<td>0.37</td>
<td>6.6</td>
</tr>
<tr>
<td>$M$ 7.0 events at $R = 70$ km for eastern Canadian sites [Atkinson, 1999]</td>
<td>Scaled in time domain to match design (median) UHS</td>
<td>2</td>
<td>0.85</td>
<td>0.44</td>
<td>6.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>0.85</td>
<td>0.40</td>
<td>6.5</td>
</tr>
<tr>
<td>Historical record</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1988 $M$ 5.9 Saguenay, Chicoutimi-Nord station, $R = 43$ km, component $124^\circ$</td>
<td>Original (not scaled)</td>
<td>–</td>
<td>–</td>
<td>0.13</td>
<td>17.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1940 $M$ 6.9 Imperial Valley, El Centro station, $R = 12$ km, N-S component</td>
<td>Original (not scaled)</td>
<td>–</td>
<td>–</td>
<td>0.35</td>
<td>24.4</td>
</tr>
</tbody>
</table>

$M$, moment magnitude; PGA, peak ground acceleration; $R$, epicentral distance; $t_D$, Trifunac duration (time interval during which 5% to 95% of the total ground shaking energy is delivered.)
Figure 2.6 Time histories of typical selected ground motions.
Figure 2.7 The 5%-damped acceleration response spectra of selected ground motions for Montréal: (a) simulated records; (b) historical records.
vibration modes. This Rayleigh damping ensures that the modal damping ratios associated with all elastic vibration modes of the analyzed structural model are mostly within the range of 1%-2% of critical. In addition, this damping model should not produce any problems of force equilibrium due to high damping in the high modes. Note that inelastic behavior adds hysteretic damping to the initial damping.

2.5 Inelastic seismic analysis results

The seismic performance of the core wall was assessed from inelastic adaptive pushover and time-history dynamic analyzes. Only results obtained from the dynamic analysis are presented in this paper. Those obtained from the pushover analysis can be found in Boivin [2006]. These results confirm the intended inelastic mechanism of each wall system at the anticipated lateral drifts and the large flexural overstrength of the core wall. Due to this overstrength, they predict little inelastic flexural behavior of the wall at design drifts and therefore a much better performance level than that corresponding to the performance level “extensive damage”, which is the level expected by the 2005 NBCC for the 2% in 50 years seismic design event and is associated with inelastic deformation at or near SFRS capacity.

The dynamic analysis was performed for probable resistance only. It was carried out using the implicit Newmark constant average acceleration integration method with a constant time step of 0.001 s and using a Newton-Raphson iteration within each time step. P-delta effects were not considered because they were found negligible for the studied structure. This is typical for walls for which higher modes control their seismic response [Tremblay et al., 2001].

The dynamic analysis results given in this section are only for a few engineering demand parameters (EDPs). The selected EDPs are wall bending moment, curvature ductility, drifts, and wall shear force. Figure 2.8 shows the predicted interstorey drift and wall force demands. These demands are presented as follows: (i) mean (M) plus or minus one standard deviation (SD) peak response (PR) to selected design UHS-compatible simulated and historical records for Montreal; (ii) mean peak response (MPR) to selected 84th percentile (84th) UHS-compatible historical records for Montreal; and (iii) MPR to selected original El Centro (OEC) record. The mean and standard deviations include the peak responses computed with the 1% CD, 2% CD, and 2% RD damping models. In Fig. 2.8, the predicted demands are compared with the design envelopes. Note that the design envelopes do not include any torsional effects as only 2D analysis is considered herein. The design
interstorey drift envelopes were determined from 2005 NBCC requirements, using an SRSS combination, while the design force envelopes are the capacity design strength envelopes determined from CSA standard A23.3-04 and its explanatory notes [CAC, 2006b]. For comparison purposes, the CSA design moment envelope is presented in Fig. 2.8.b for a probable resistance, though it is based on a factored resistance in the code. The linear moment envelope suggested in CAC [1995] is also shown in Fig. 2.8.b. The CSA design shear envelope shown in Fig. 2.8.c does not include the dynamic magnification due to inelastic effects of higher modes. Note that this envelope is governed by \( V_p \) over the assumed base hinging region and by the shear envelope determined in accordance with the new CSA shear strength requirement for all elevations above that of the hinging region.

### 2.5.1 Overall behavior and flexural demand

For either seismic loading direction, little flexural yielding is predicted in the core wall when subjected to design-level ground motions, and substantial yielding, without exceeding the ultimate flexural capacity, is predicted when subjected to 84th-level and OEC ground motions.

For north-south seismic loading, the simulations with the cantilever wall model generally predict a plastic hinge at the wall base and, in some cases, a few additional hinges at the middle and upper storeys for the design-level, 84th-level, and OEC excitations, as indicated in Table 2.4 for a 2% RD (note that similar predictions are obtained with the 1% and 2% CDs). The same simulations conducted with EFiCoS do not predict any yielding above the assumed base hinging region for a design-level excitation but do predict onset of yielding in the outermost reinforcing bars at mid-storey for the above-design-level excitations, as shown in Fig. 2.9. Based on this figure, no concrete damage in compression is predicted even at the 84th-level excitation, but tensile cracking over a large part of the wall is predicted. Based on EFiCoS predictions, it appears that the hinges at the middle and upper storeys predicted by the RUAUMOKO cantilever wall model are more a modeling issue than a design issue related to lumped plasticity modeling. This statement is reinforced by the flexural predictions obtained with the RUAUMOKO model of the north-south building, where the inelastic action in the wall is constrained at its base for the three levels of excitation, as indicated in Table 2.4. This suggests that the modeling approach adopted and the added stiffness from structural components not part of the SFRS may have a strong influence on hinge formation predictions for multistorey cantilever walls. Nevertheless, the above predictions indicate that the capacity design moment envelope determined from the new CSA standard A23.3-04 provision has prevented the formation
Figure 2.8 Inelastic time-history dynamic analysis results obtained from RUAUMOKO structural models: (a) drift; (b) flexure; (c) shear.
Table 2.4  Predicted plastic hinges in wall systems for Rayleigh damping with 2% in 1st and 12th lateral vibration modes (2% RD).

<table>
<thead>
<tr>
<th>Structural model</th>
<th>Design UHS-compatible records</th>
<th>84th UHS-compatible records</th>
<th>El Centro (Original)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M6@30km</td>
<td>M7@70km</td>
<td>Saguenay</td>
</tr>
<tr>
<td></td>
<td>location</td>
<td>$\mu_\phi$</td>
<td>location</td>
</tr>
<tr>
<td>Cantilever wall</td>
<td>None</td>
<td>-</td>
<td>Base</td>
</tr>
<tr>
<td></td>
<td>S8</td>
<td>3.4</td>
<td></td>
</tr>
<tr>
<td>N-S building</td>
<td>Base</td>
<td>1.3</td>
<td>Base</td>
</tr>
<tr>
<td>Coupled wall</td>
<td>Beams</td>
<td>2.5</td>
<td>Beams</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: S6-S9 denote storey level (with S1 as the base up to S13 as the penthouse). $\mu_\phi$, curvature ductility.

$^a$ Base of walls.
of unintended plastic hinges, even for an excitation level significantly above design level. Actually, Fig. 2.8b shows that, in general, the CSA design envelope conservatively estimates the predicted wall moment demands for the three levels of excitation. This figure also shows that the linear probable envelope suggested in CAC [1995] underestimates the predicted demands at the middle and upper storeys. This shows once more that this envelope is inadequate to capture the higher mode contribution in the flexural response.

![Figure 2.9](image)

Figure 2.9  Damage predicted by EFicCoS in cantilever wall system under 84th-level Saguenay motion for a 2% RD: (a) concrete damage in compression; (b) concrete damage in tension; (c) plasticity in steel rebars (Concrete damage: blue (dark grey in print version) = 0%; red (light grey in print version) = 100%; Rebar plasticity: blue = no; red = yes)

For east-west seismic loading, the simulations with the coupled wall model predict hinging solely at beam ends when subjected to design-level motions and at both beam ends and wall base when subjected to 84th-level and OEC motions. Even under the above-design-level motions, the mean beam curvature ductility demand remains lower than the ultimate curvature ductility, which corresponds to the inelastic rotational capacity of 0.04 rad. specified in CSA standard A23.3-04 for diagonally reinforced coupling beams. Figure 2.8b shows that the predicted wall moment demands are conservatively estimated by the 2004 CSA design envelope (which was determined for the compression wall), except when a wall acts
as a tension wall under the 84th-level and OEC excitations. In these cases, the predicted flexural demand at upper storeys exceeds the design envelope.

It is of interest to note that the RUAUMOKO model of the complete north-south building predicts plastic hinges only in the wall system when subjected to the design-level motions. Additional plastic hinges are predicted mainly in the base columns of all frames and the spandrel beams of the exterior frame when subjected to above-design-level motions.

### 2.5.2 Overall and interstorey drifts

Figure 2.8a shows that, for either seismic loading direction, the interstorey drift demands predicted with the design-level motions are less than or equal to 0.5%, while those predicted with the 84th-level and OEC motions range between 0.5% and 1.5%. These values are largely less than the NBCC limit of 2.5% for the studied building. Although not shown, the mean predicted overall drift demands are less than 0.3% and 0.7% for the design-level and above-design-level motions, respectively. Figure 2.8a shows that the NBCC design envelope for the cantilever wall direction underestimates the predicted drift demand when the structure is subjected to design-level motions. For both directions, the design envelopes significantly underestimate the drift demands predicted with the above-design-level motions. Obviously any large drifts resulting from local yielding cannot be captured by the NBCC envelopes, since they were determined from linear analysis.

### 2.5.3 Shear demand

For either seismic loading direction, Fig. 2.8c shows that the wall shear force demand predicted with the design-level motions significantly exceeds the CSA design envelope, and hence $V_p$, though the shape of both the demand and the envelope over the wall height is almost the same. This figure also shows that the wall shear predictions obtained with the cantilever wall model and the north-south building model are very similar, though slightly less with the building model. This means that the added stiffness from structural components not part of the SFRS has a negligible effect on the wall shear predictions. From Fig. 2.8c, it is noted that the OEC motion produces a larger base shear demand on the cantilever wall than the 84th-level motions, even though its high-frequency acceleration content is much less significant, as illustrated in Fig. 2.7. An opposite result, however, is observed in Fig. 2.8c for the coupled wall direction, particularly when the wall acts as a tension wall. These observations suggest that spectrum-compatible motions may not represent conservative input, unlike commonly assumed, for wall shear predictions.
The ratio of the predicted wall shear force demand to the CSA design shear envelope at each storey is defined as the dynamic shear magnification factor $\beta_V$. Tables 2.5 and 2.6 give this factor for the cantilever and coupled wall systems, respectively. In these tables, the $\beta_V$ value at the wall base and the average value over all storeys (AOS) are given for the MPR and M–SD PR to design-level motions and the MPRs to OEC and 84th-level motions. These tables show that, for the M–SD demand predicted from the design-level motions, the base and AOS $\beta_V$ values are about 1.5 for the cantilever system and about 1.4 and 1.5 for the coupled wall system. The $\beta_V$ values are not larger than 1.9 for the above-design-level excitations. For comparison purposes, the value of New Zealand dynamic shear magnification factor, $\omega_V$, for the studied building is 1.7.

### Table 2.5 Dynamic shear magnification factor, $\beta_V$, for cantilever wall system (from building model)

<table>
<thead>
<tr>
<th></th>
<th>Design UHS-compatible records (MPR)</th>
<th>84th UHS-compatible records (MPR)</th>
<th>Original El Centro record (MPR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AOS</td>
<td>1.29</td>
<td>1.47</td>
<td>1.60</td>
</tr>
<tr>
<td>Base</td>
<td>1.33</td>
<td>1.48</td>
<td>1.57</td>
</tr>
</tbody>
</table>

### Table 2.6 Dynamic shear magnification factor, $\beta_V$, for coupled wall system (maximum of both tension and compression walls)

<table>
<thead>
<tr>
<th></th>
<th>Design UHS-compatible records (MPR)</th>
<th>84th UHS-compatible records (MPR)</th>
<th>Original El Centro record (MPR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AOS</td>
<td>1.28</td>
<td>1.48</td>
<td>1.69</td>
</tr>
<tr>
<td>Base</td>
<td>1.26</td>
<td>1.39</td>
<td>1.86</td>
</tr>
</tbody>
</table>

2.6 Discussion

Based on the analysis results previously presented, the overall seismic performance in flex­ure to be expected for the core wall structure under possible design-level ground motions is much better than the performance level “extensive damage” (equivalent to “near collapse” in the SEAOC Vision 2000 committee document [SEAOC, 1995]) expected by the 2005 NBCC for the 2% in 50 years seismic design event. Actually, as damage increases
2.6. DISCUSSION

with increasing inelastic lateral deformation, the low level of inelastic action predicted in the wall structure for the design-level excitation indicates that the structure should be slightly damaged. Even for the 84th-level and OEC excitations, the predicted lateral deformation demand on the structure is less than the deformation capacity. This better than expected performance is mainly due to the large flexural overstrength resulting from the excess strength at the wall base arising from the required minimum reinforcement. This excess strength then inhibits the intended large inelastic deformation of the ductile wall structure under the design earthquake and the intended force reduction assumed for design. This means that excess strength in flexure should be avoided as much as possible in the assumed base hinging region of a ductile wall.

It is important to add that the predicted good flexural performance of the studied structure is also due to an adequate prevention of unintended plastic hinge formation in the walls. This suggests that the new capacity design method of CSA standard A23.3-04 for the flexural strength design of ductile walls is adequate to constrain the inelastic mechanism of the walls at their intended base hinging region. Note that coupling beams provide an additional inelastic mechanism in ductile coupled wall systems.

Despite the predicted good flexural performance of the wall structure, the predicted shear demand for the design-level excitation significantly exceeds the CSA design shear envelope. This suggests a potential shear failure, which would prevent the wall structure to laterally deform in a ductile manner. In such cases, the seismic performance of the structure might be closer to "extensive damage" than what is predicted in flexure. This scenario is not so unrealistic because the current predictions do not account for shear increase due to torsional effects.

One could explain the exceeding shear predictions for the design-level excitation by the linear shear assumption used to model the shear behavior of the wall structure. Laboratory tests [Oesterle et al., 1977] showed that flexural yielding at the base hinging region of a wall triggers shear yielding behavior, even if shear strength design is based on capacity design principles. Another explanation could be the dynamic magnification due to inelastic effects of higher vibration modes, since the CSA design envelope does not account for this magnification, which should increase with ductility demand as the relative contribution of higher modes increases [Priestley, 2003; Seneviratna and Krawinkler, 1994]. Actually, based on Tables 2.5 and 2.6, scaling up the CSA design envelope with the dynamic shear magnification factor $\omega_v$ of 1.7 for the studied building produces much better wall shear estimates for motions at design level and even above. However, none of
the above explanations stand up because very little inelastic action, and hence ductility
demand, is predicted in the wall structure for the design-level excitation.

Figure 2.10 illustrates a reason for the exceeding shear demand under the design-level
excitation. Actually, the predictions obtained from the inelastic dynamic analysis are
based on realistic 1%-2% damping ratios, whereas the CSA design shear envelope is based
on the common 5% damping, as it was obtained by scaling up the NBCC design shear
envelope determined from the response spectrum method. This damping overestimation
produces a significant underestimation of the higher mode contribution in the NBCC
design forces due to lower spectral accelerations ($S_a(T)$), as illustrated in Fig. 2.10. As
a result, the CSA design envelope underestimates the predicted wall shear demand, even
though the wall response is almost elastic under the design-level excitation. The use of a 5%
damping for the inelastic dynamic analysis would have hidden the shear underestimation
problem for the design-level excitation, as shown in Fig. 2.11. Of course, the CSA design
shear envelope will underestimate any excitations above design level, as shown in Fig. 2.11,
because shear demand, including inelastic dynamic magnification effects, increases with
an increase in ground motion intensity [Amaris, 2002]. The previous observations suggest
then that the 5% damped spectral accelerations ($S_a(T)$) prescribed by the 2005 NBCC for
seismic design can underestimate the higher mode responses of walls sensitive to higher
mode effects and, as a result, that the capacity design method prescribed by CSA standard
A23.3-04 can be inadequate for estimating the shear demand on such walls, even when
their behavior is lightly inelastic.

In order to verify this statement, simplified acceleration response spectra similar to the
NBCC design spectrum were determined, for 1% and 2% dampings, from the mean re­
sponse spectra of all selected simulated records for Montréal, as shown in Fig. 2.10. Al­
though not shown, these simplified spectra lie between the mean and the M–SD design
spectra proposed by Atkinson and Pierre [2004] for 1% and 2% damping. Using the sim­
plified spectra as input in the linear response spectrum method, new NBCC and CSA
seismic design envelopes were determined for 1% and 2% damping. Although they are not
shown in this paper, these envelopes are very good or conservative estimates of the de­
mands predicted from the inelastic time-history analysis for design-level motions and the
same damping values. The new CSA design shear envelope for the walls, however, match
fairly well or underestimate the M–SD demand predicted from the inelastic time-history
analysis, particularly in the cantilever wall direction. Table 2.7 gives the probable base
shear ($V_p$) for the cantilever wall system determined from $S_a(T)$ values based on 1%, 2%
and 5% (code value) damping. This table indicates that, as $V_f$ (and $M_f$) considerably
2.6. DISCUSSION

Figure 2.10 Mean and simplified acceleration response spectra obtained from all selected simulated records for Montréal. $T_1 - T_3$, 1st–3rd lateral vibration periods of the studied building.

Figure 2.11 Shear responses of the cantilever wall model for 5% damping
increases with decreasing damping, $\gamma_p$ decreases, given that $M_p$ at the wall base remains constant. This results in $V_p$ increasing by only 6% and 16% as damping drops from 5% to 2% and 1%, respectively. Based on Table 2.5, these increases are not enough to adequately estimate the predicted shear demand on the cantilever wall system. This shows a deficiency of Eq. (2.3) to suitably estimate the shear demand on walls whose seismic response is dominated by higher modes, even though this response is almost elastic. This deficiency reinforces the need for CSA standard A23.3-04 to provide an adequate capacity design method, taking into account the inelastic magnification effects of higher modes, for the shear strength design of ductile RC walls.

<table>
<thead>
<tr>
<th>Damping for $S_a(T)$</th>
<th>$\gamma_p$</th>
<th>$V_f$ (kN)</th>
<th>$V_p$ (kN)</th>
<th>$V_p/V_p^{5%}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>5.00</td>
<td>2062</td>
<td>10300</td>
<td>-</td>
</tr>
<tr>
<td>2%</td>
<td>3.71</td>
<td>2937</td>
<td>10915</td>
<td>1.06</td>
</tr>
<tr>
<td>1%</td>
<td>3.24</td>
<td>3691</td>
<td>11972</td>
<td>1.16</td>
</tr>
</tbody>
</table>

### 2.7 Conclusion

In this work, a 12-storey ductile concrete core wall located in Montréal was designed according to the 2005 NBCC and the CSA standard A23.3-04 for seismic loading and analyzed in the nonlinear regime. The analysis results obtained in this work indicate that the overall seismic performance in flexure to be expected for the core wall structure under possible design-level ground motions is much better than the performance level "extensive damage" expected by the 2005 NBCC for the 2% in 50 years seismic design event. However, there is a risk of shear failure of wall members due to an underestimation at the design stage of the seismic wall shear demand.

This work suggests the following main conclusions with regard to seismic design with the 2005 NBCC and CSA standard A23.3-04 for mid-rise ductile RC shear walls:

1. Unlike the linear probable moment method suggested in CAC [1995], the new method prescribed by CSA standard A23.3-04 for determining the capacity design moment
envelope for ductile walls may provide conservative estimates and hence may prevent unintended plastic hinge formation.

2. The 5% damped spectral accelerations ($S_a(T)$) prescribed by the 2005 NBCC underestimate the higher mode responses of walls whose seismic response is dominated by higher modes.

3. The shear strength requirements prescribed by CSA standard A23.3-04 can produce capacity design shear envelopes that significantly underestimate the shear demand on such walls, even when the wall behavior is slightly inelastic and the NBCC design forces are determined from $S_a(T)$ values based on realistic damping values for these walls. This suggests that the new design codes may be inadequate to prevent a shear failure in such walls under design earthquake.

4. A capacity design method for shear strength design is required for CSA standard A23.3-04. Meanwhile, scaling up the CSA design shear envelope by the New Zealand dynamic shear magnification factor $\omega_v$ seems to be a reasonable approach for estimating wall shear demand.

As this work is based on two-dimensional (2D) simulations of a single typical RC core wall structure that, moreover, has substantial flexural overstrength, more analysis is needed to reinforce the previous conclusions.

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CHAPTER 3

Parametric study of ductile cantilever walls

Foreword

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Journal : Canadian Journal of Civil Engineering

Title : Seismic force demand on ductile RC shear walls: Part 1 - Parametric study

Titre français : Demande sismique en force sur des murs de refend ductiles en béton armé: partie 1 - étude paramétrique

Paper’s contribution to this project : This paper contributes to the thesis by showing from an extensive parametric study the influence of various parameters on the higher mode amplification effects, and hence on the seismic force demand, in isolated regular ductile concrete cantilever walls designed with the 2010 National building code of Canada (NBCC) and the 2004 Canadian Standards Association (CSA) standard A23.3. Also it shows that the capacity design methods prescribed by the 2004 CSA standard A23.3 can produce capacity design envelopes that fail to conservatively estimate the seismic moment and shear force demand on such walls under design-level ground motions. This raises up the necessity of new capacity design methods for the CSA standard A23.3 for flexural and shear strength design of these walls.
Abstract: A parametric study of regular ductile reinforced concrete (RC) cantilever walls designed with the 2010 National building code of Canada and the 2004 Canadian Standards Association (CSA) standard A23.3 for Vancouver is performed in order to investigate the influence of the following parameters on the higher mode amplification effects, and hence on the seismic force demand: number of storeys, fundamental lateral period ($T$), site class, wall aspect ratio, wall cross-section and wall base flexural overstrength ($\gamma_w$). The study is based on inelastic time-history analyses performed with a multilayer beam model and a smeared membrane model accounting for inelastic shear-flexure-axial force interaction. The main conclusions are that (i) $T$ and $\gamma_w$ are the studied parameters affecting the most dynamic shear amplification and seismic force demand, (ii) the 2004 CSA standard A23.3 capacity design methods are inadequate, and (iii) a single-plastic hinge design may be inadequate and unsafe for regular ductile RC walls with $\gamma_w < 2.0$.

Key words: Parametric study, 2004 CSA standard A23.3, ductile concrete cantilever wall, capacity design, higher mode amplification effects, seismic force demand

Résumé: Une étude paramétrique de murs ductiles réguliers en béton armé dimensionnés avec le Code national du bâtiment 2010 du Canada et la norme CSA A23.3 2004 pour Vancouver est réalisée afin d'étudier l'influence des paramètres suivants sur les effets d'amplification des modes supérieurs, et donc sur la demande sismique en force: le nombre d'étages, la période fondamentale de vibration latérale ($T$), la classe du site, l'élancement du mur, la section du mur et la surcapacité flexionnelle à la base du mur ($\gamma_w$). L'étude est basée sur des analyses dynamiques inélastiques réalisées à l'aide d'une modélisation par poutres multi-couches et une modélisation par membranes considérant l'interaction inélastique force axiale-flexion-cisaillement. Les conclusions principales sont que (i) $T$ et $\gamma_w$ sont les paramètres étudiés ayant le plus d'influence sur l'amplification dynamique en cisaillement et sur la demande sismique en force, (ii) les méthodes de dimensionnement à la capacité de la norme CSA A23.3 2004 sont inadéquates, and (iii) une conception à rotule plastique unique peut être inadéquate et non sécuritaire pour des murs ductiles réguliers en béton armé dont $\gamma_w < 2.0$.

Mots clés: Étude paramétrique, Norme CSA A23.3 2004, mur ductile en béton armé, dimensionnement à la capacité, effets d'amplification des modes supérieurs, demande sismique en force
3.1 Introduction

In order 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. The capacity design aims to ensure that the inelastic mechanism develops as intended and no undesirable failure modes occur. Since the 1984 edition, this design approach is implemented in the Canadian Standards Association (CSA) standard A23.3 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. The implemented capacity design requirements intend to constrain the inelastic mechanism of a regular wall at the expected base plastic hinge. This design is referred to as single plastic-hinge (SPH) design.

Despite the large improvements made to the seismic design provisions of the 2005 edition of the National building code of Canada (NBCC) [NRCC, 2005] and the 2004 edition of the CSA standard A23.3 (A23.3-04) [CSA, 2004], these codes can still produce inadequate and potentially risky seismic designs of regular multistorey ductile RC cantilever or coupled wall structures whose seismic force response is dominated by lateral modes of vibration higher than the fundamental lateral mode. The recent work of Boivin and Paultre [2010] shows that for such walls the capacity design shear envelope determined from these codes to prevent shear failure can largely underestimate the shear force demand under design-level ground motions, even when the wall response is slightly inelastic. This underestimation results from a deficiency of these codes to account for dynamic amplification effects, in the elastic and inelastic regimes, due to higher lateral modes. Boivin and Paultre identified two sources for this underestimation: (i) the 2005 NBCC spectral accelerations underestimate the higher mode responses of the walls because their traditional 5% damping overestimates actual damping (about 2% on average) and hence reduces the higher mode responses; and (ii) the capacity design methods prescribed by CSA standard A23.3-04 for shear strength design do not account for the dynamic amplification of shear forces due to inelastic effects of higher modes. No such underestimation problem in flexure was reported by Boivin and Paultre, likely because of the large flexural overstrength of the wall system studied by the authors. However, the capacity design method for flexural strength design prescribed by CSA standard A23.3-04 to prevent unintended plastic hinges above the base hinging region is not free from such problem because its formulation based on amplified elastic forces is tributary of the analysis method, static or dynamic, used
to derive these forces. The minor changes made to the seismic provisions of the 2010 NBCC [NRCC, 2010] do not address these issues.

In this regard, this work proposes for CSA standard A23.3 new capacity design methods, considering higher mode amplification effects, for determining, for a SPH design, capacity design envelopes for flexural and shear strength design of regular ductile RC cantilever wall structures used as SFRS for multistorey buildings. This objective is achieved first by investigating from a parametric study the influence of various parameters on the higher mode amplification effects, and hence on the seismic force demand, in these walls, and second by deriving from the investigation results new capacity design methods for determining adequate capacity design shear and moment envelopes for such walls over their height. Note that the work focusses solely on cantilever walls because higher mode amplification effects are usually much more important in cantilever walls than in coupled walls.

This paper presents the first part of this work, that is, the parametric study. The paper is broken down into first a short literature review on higher mode amplification effects in RC walls in order to identify parameters that can have a significant influence on these effects, followed by an outline of the methodology adopted for the parametric study and finally, a presentation and a discussion on the results.

### 3.2 Review on higher mode effects in RC walls

The seismic force response of RC shear wall structures used as SFRS for multistorey 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 [Priestley, 2003; Sangarayakul and Warnitchai, 2004; Seneviratna and Krawinkler, 1994]. This dynamic amplification is associated to inelastic effects of higher lateral modes.
3.2. REVIEW ON HIGHER MODE EFFECTS IN RC WALLS

Research on higher mode amplification effects in RC walls has mainly focussed on the estimation of seismic shear forces in cantilever walls designed for a SPH at the base. The research has led to several relations for estimating seismic shear force demand on cantilever walls, particularly the base shear force. In most of these relations, the shear force corresponding to first-mode response at flexural capacity is increased by a dynamic amplification factor accounting for the elastic and inelastic effects of higher modes. In the following, a review of these dynamic factors is conducted in order to identify the parameters having a significant influence on the higher mode effects in cantilever walls and bring out the trends of their influence.

The pioneering work on higher mode effects in RC cantilever walls is that of Blakeley et al. [1975], which is based on inelastic dynamic analyses of isolated walls designed for a SPH at the base. The authors found that, at some instants of the response, the resultant lateral inertia force of a predominantly higher-mode response can be located much lower along the wall height than that of a first-mode response, producing a base shear increase and a base moment reduction. Moreover, they found that this base shear amplification increases with the fundamental lateral period of vibration, $T_1$, of the structure and the ground motion intensity, given the essentially elastic response of higher modes, and decreases with the flexural overstrength at the wall base. In addition, they highlighted the possibility of plastic hinge formation at levels above the base. The main outcome from this work is the well-known dynamic shear magnification factor for cantilever walls, $\omega_v$, implemented in the New Zealand concrete design standard [NZS, 2006] to magnify the static shear force corresponding to first-mode response at flexural capacity:

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

where $N$ is the number of storeys of the building. This factor accounts for the elastic and inelastic effects of higher modes. Iqbal and Derecho [1980] proposed similar factors, based on $T_1$ this time, for shear and moment strength design.

Several works [Aoyama, 1987; Ghosh and Markevicius, 1990; Kabeyasawa and Ogata, 1984] attempted to estimate the maximum seismic shear force $V_{\text{max}}$ at the base of a SPH RC cantilever wall, isolated or part of a wall-frame structure, by adding to the wall base shear force corresponding to first-mode response at flexural capacity, $V_{fy}$, a force corresponding to higher mode responses, $V_{hm}$. All these works proposed a relation whose
format is in essence the same and has shown to be in good agreement with experimental results [Eberhard and Sozen, 1993]. This relation can be expressed as follows:

\[
V_{\text{max}} = V_{1y} + V_{hm} = M_y/0.67H + D_m W A_g \\
= \omega_{mv} V_{1y} \\
\omega_{mv} = 1 + D_m W A_g / V_{1y}
\] (3.2)

(3.3)

where \(\omega_{mv}\) is a dynamic base shear amplification factor, \(M_y\) is the wall bending strength at the base determined from an inverted triangular force distribution over the entire height \(H\) of the wall, \(W\) is the total weight of the structure, \(A_g\) is a peak acceleration coefficient, expressed as a ratio of gravitational acceleration, taken as either the peak acceleration or an effective peak acceleration of the input ground motion, and \(D_m\) is a coefficient that for some is constant and equal to 0.25 [Ghosh and Markevicius, 1990], and for others increases with the number of storeys \((N)\) of the structure [Aoyama, 1987; Kabeyasawa and Ogata, 1984] and with the design displacement ductility ratio, \(\mu_\Delta\) [Seneviratna and Krawinkler, 1994]. For \(N \geq 30\), Seneviratna and Krawinkler [1994] observed that \(D_m\) becomes independent from the ductility ratio and converges toward a value of about 0.34, as found by Aoyama [1987]. The dependence of \(\omega_{mv}\) (Eq. (3.3)) on \(N\) is in line with that of \(\omega_v\) (Eq. (3.1)). However, \(\omega_{mv}\) directly captures more parameters having an influence on higher mode effects than \(\omega_v\). This deficiency of \(\omega_v\) in capturing causative parameters can result in significant underestimations of dynamic shear amplification in walls [Keintzel, 1990; Priestley and Amaris, 2003; Seneviratna and Krawinkler, 1994].

From inelastic dynamic analyses of isolated SPH RC cantilever walls designed from a direct displacement-based design (DDBD) method, Priestley [2003] derived the following dynamic base shear amplification factor, \(\omega_\star\), which aims to amplify the base shear determined with the DDBD method and also attempts to fix the underestimation issue of \(\omega_v\) (Eq. (3.1)):

\[
\omega_\star = 1 + B_T \mu_\Delta / \phi_\alpha \\
B_T = 0.067 + 0.4 (T_1 - 0.5) \leq 1.15 \quad (T_1 \geq 0.5 \text{s})
\] (3.4)
(3.5)
where $\phi_o$ is the flexural overstrength factor at the wall base. Figure 3.1 illustrates Eq. (3.4) for $\mu_\Delta = 5.6$ and four $\phi_o$ values, and compares it with Eq. (3.1). Figure 3.1 shows a considerable influence of $\phi_o$ on dynamic base shear amplification and a large difference between the two relations, in part because of the different design methods to which they are associated. A comparison between Eqs. (3.4) and (3.3) shows similarities, knowing that $D_m$ in some extent depends on $T_1$ and that $V_{1e}$ can be expressed as $\phi_o V_{1e}/\mu_\Delta$ where $V_{1e}$ is the elastic base shear force associated to first lateral mode.

For RC walls designed for high ductility, the 2004 edition of the Eurocode 8 (EC8) [CEN, 2004] requires that the design shear force diagram determined from linear analysis be amplified by the following dynamic shear amplification factor, $\varepsilon$, which is based on the formula proposed by Keintzel [1990]:

$$\varepsilon = q \cdot \sqrt{\left(\frac{\gamma_{Rd}}{q} \cdot \frac{M_{Rd}}{M_{Ed}}\right)^2 + 0.1 \left(\frac{S_e(T_e)}{S_e(T_1)}\right)^2} \quad (3.6)$$

with $1.5 \leq \varepsilon \leq q$, where $q$ is the behavior factor (equivalent to $\mu_\Delta$) used for design, $M_{Rd}$ is the design flexural resistance at the wall base, $M_{Ed}$ is the design bending moment at the wall base, $\gamma_{Rd}$ is the factor to account for overstrength due to strain hardening of the reinforcement (may be taken as 1.2), $T_1$ is the fundamental lateral period of the structure, $T_e$ 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 $T$. Eq. (3.6) was derived considering only the first two lateral vibration modes. The first term within the square root corresponds to the shear force likely to develop at flexural
capacity under a first-mode response, while the second term corresponds to the shear increase due to higher mode effects. Note that, unlike CSA standard A23.3-04, which is silent on dynamic shear amplification in moderately ductile (MD) walls, the 2004 EC8 recommends $\varepsilon = 1.5$ for these walls. Various works [Linde, 1998; Priestley and Amaris, 2003; Rutenberg and Nsieri, 2006] indicated that Eq. (3.6) can significantly overestimate the dynamic base shear amplification in short-period ductile walls while underestimating it in long-period ones. In an attempt to fix that problem, Rutenberg and Nsieri [2006] proposed the following formulas for the 2004 EC8 for determining the seismic design base shear force, $V_{db}$, for ductile walls:

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

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

where $\varepsilon^*$ is a dynamic base shear amplification factor. Eq. (3.7) is based on the observation that dynamic amplification of the base shear force increases quite linearly with $T_1$ and $q$. The $\varepsilon^*$ factor is compared to Eqs. (3.1) and (3.4) in Fig. 3.1 for $q = 5.6$.

Since the 2005 edition, the NBCC has introduced a new factor, $M_v$, in the calculation of the code-specified base shear force to account explicitly for the dynamic magnification of base shear due to elastic effects of higher modes. Table 3.1 gives the $M_v$ values specified in the 2010 NBCC for cantilever walls. A ratio $S_a(0.2)/S_a(2.0) < 8.0$ is typical for the western Canadian regions, where earthquake ground motions have primarily a low frequency content, and a ratio $S_a(0.2)/S_a(2.0) \geq 8.0$ is typical for the eastern Canadian regions, where ground motions have principally a high frequency content. Once again, Table 3.1 shows, as Eqs. (3.3) and (3.6), that, in addition to the fundamental lateral period of the building ($T_a$), the input earthquake motion, and hence the seismic zone, has an influence on higher mode effects, such as suggested also by Filiatrault et al. [1994] for selecting their proposed shear reduction factor.

<table>
<thead>
<tr>
<th>$S_a(0.2)/S_a(2.0)$</th>
<th>$T_a \leq 1.0$</th>
<th>$T_a = 2.0$</th>
<th>$T_a \geq 4.0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 8.0</td>
<td>1.0</td>
<td>1.2</td>
<td>1.6</td>
</tr>
<tr>
<td>$\geq 8.0$</td>
<td>1.0</td>
<td>2.2</td>
<td>3.0</td>
</tr>
</tbody>
</table>
3.2. REVIEW ON HIGHER MODE EFFECTS IN RC WALLS

Based on the previous review on dynamic shear amplification factors, it can be concluded that the parameters affecting the most the higher mode amplification effects in isolated RC cantilever walls are the fundamental lateral period \( (T_1) \) of the structure, and/or the number of storeys \( (N) \), the design displacement ductility ratio \( (\mu_D) \), the flexural overstrength at the wall base and the ground motion intensity and frequency content, which implies the seismic zone which is characterized by its seismic hazard and site conditions. From all reviewed relations of dynamic shear amplification factor, Eq. (3.3) appears the most appropriate to directly capture the main parameters affecting higher mode effects.

It is important to point out, however, that the above observations and relations for estimating dynamic shear amplification were derived from two-dimensional (2D) inelastic time-history analyses (ITHAs) performed with generally lumped plasticity beam elements where flexural deformation was modelled with bilinear hysteresis rules and shear deformation was modelled linearly elastic or with a nonlinear shear spring uncoupled from flexural and axial deformations. This simple modeling tends to largely overestimate shear predictions because important nonlinear physical phenomena, such as cracking, shear-flexure-axial force interaction and strength decay, that occur in actual laterally deformed RC walls are not taken into account. Moreover, such modeling could not capture the inelastic shear deformation differences between different wall cross-sections. In addition, the ITHAs were often carried out with strong historical ground motions of western U.S. that are not necessarily representative of the magnitude-distance ranges and tectonic environment that cause the seismic hazard of the main Canadian seismic regions for the design probability level. These important modeling deficiencies and input earthquake differences highlight the limitations of the presented observations and relations. This indicates the need for further investigation on the parameters influencing higher mode effects in isolated RC walls and for more sophisticated and representative simulations to get realistic estimates for Canadian regions.

It is also important to add that the list of parameters influencing the higher mode amplification effects in RC shear wall systems is not limited to those affecting isolated cantilever walls. System-related parameters, such as the beam-to-wall strength ratio in coupled walls [Munshi and Ghosh, 2000], the sequence of hinge formation [Rutenberg and Nsieri, 2006] and the relative inelastic shear deformation [Adebar and Rad, 2007] between the walls composing a wall system, and the frame-to-wall stiffness ratio in dual systems [Rutenberg and Nsieri, 2010], can also play a significant role on dynamic shear amplification. However, these parameters are not addressed in this paper.
3.3 Parametric study methodology

The parametric study aims to investigate the influence of various parameters on the higher mode amplification effects and hence on the seismic force demand on regular ductile RC cantilever walls. In this regard and based on the outcomes from the previous review, the methodology adopted in this work for the parametric study is as followed:

1. Selecting the parameters to be studied and assigning them a value range;
2. Designing and detailing for seismic forces each studied wall case with the 2010 NBCC and CSA standard A23.3-04 to meet the parameter values associated to that case;
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;

Each stage is detailed in the following sections.

3.3.1 Studied parameters

From the parameters identified in Section 3.2, the following were considered for the parametric study: number of storeys \( N \), fundamental lateral period \( T \), design displacement ductility ratio \( \mu_\Delta \), flexural overstrength at the wall base \( \gamma_w \), wall aspect ratio \( A_r \), wall cross-section \( \text{WCS} \), seismic zone and site class \( \text{SC} \). The \( \mu_\Delta \) ratio and the seismic zone are the only two parameters fixed for the study. In the 2010 NBCC, the \( \mu_\Delta \) 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 \( R_dR_o = 3.5 \cdot 1.6 = 5.6 \). The seismicity of the city of Vancouver located on the Canadian West coast was selected for the study because this city has the highest urban seismic risk in Canada. Its high seismic hazard is representative of that of western Canadian cities and ductile RC walls are the preferred SFRS in this region. Figure 3.2 shows the 2010 NBCC design spectra for Vancouver for different site classes. Although the earthquake ground motions spectra of the West coast have typically a lower high frequency content than those of eastern Canada, their motion intensity is in general much higher. Consequently, this gives a better control on flexural overstrength \( \gamma_w \) because seismic design is further governed by the design forces than by the required minimum reinforcement.

Table 3.2 gives the values considered for the studied varying parameters. The \( N \) values range from 5 to 40. For each \( N \) value, two \( T \) values were selected. Their selection is based on in-situ measurements, shown in Fig. 3.3, of fundamental lateral periods of multistorey...
3.3. PARAMETRIC STUDY METHODOLOGY

Figure 3.2 2010 NBCC (5%-damped) design spectra for Vancouver for different site classes (A: hard rock; B: rock; C: very dense soil and soft rock; D: stiff soil; E: soft soil)

Figure 3.3 Measured fundamental lateral periods vs number of storeys of RC wall buildings compared to 2010 NBCC empirical period \( T_a \) (Measurements are from Goel and Chopra [1998]; Lee et al. [2000]; Ventura et al. [2005]; Kim et al. [2009] and Gilles [2010])

RC wall buildings. Using a mean storey height of 3.5 m, Fig. 3.3 shows that most of these measurements are within the range defined by \( T_a \), the empirical fundamental period specified by the 2010 NBCC for wall buildings, and \( 2T_a \), the upper bound specified by the NBCC for seismic design of such structures. The selected \( T \) values are approximately equal to or within these limits and range from 0.5 s to 4.0 s, which is the minimum period of the constant acceleration region of the design spectra (see Fig. 3.2). Four different WCSs were considered: rectangular (RT), barbell-shaped (BB) and two I-shaped ones (I1 and H1), as illustrated in Fig. 3.4. For all studied cases, WCSs were bent about their strong
axis. For a given $N$, a typical WCS was used except for $N = 20$ where the influence of the four different WCSs on the wall response was studied. In the elastic regime, for a given $T$, changing $A_r$ does not affect the higher mode amplification effects because the latter are controlled by $T$. In the inelastic regime, however, the increase of higher mode contribution as $T$ lengthens because of base yielding may likely result into larger dynamic shear amplifications for slender walls. The selected $A_r$ values were obtained by changing the wall length ($l_w$) and by keeping constant the storey height, which is 3.5 m (3.0 m for three cases). This change results in different plastic hinge heights at the wall base as this height is assumed proportional to $l_w$. The flexural overstrength at the wall base, $\gamma_w$, is calculated as the ratio of the nominal moment resistance ($M_n$) and the design moment ($M_f$) at the wall base. As suggested by Eq. (3.4) ($\phi_o \equiv \gamma_w$), $\gamma_w$ reduces $\mu_\Delta$ resulting in an effective $\mu_\Delta$, which provides a better estimate of the expected displacement ductility demand on a cantilever wall and hence, of the likely dynamic amplification levels. Larger is $\gamma_w$, lower should be the dynamic shear amplification in the inelastic regime, as shown in Fig. 3.1. The considered $\gamma_w$ 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 be controlled by the elastic shear forces, which is $5.6/1.3 \approx 4.3$. As indicated in Table 3.2, four of the six site classes (SCs) defined in the 2010 NBCC were considered for the study. As shown in Fig. 3.2, the design spectra associated to the selected site classes enable to account for the possible soil amplification effects, excluding those associated to the unclassified site class F (other soils). The site class effects were studied only for $N \leq 10$ because soil amplification effects generally reduce with increasing $T$, assuming a soil, even a soft one, largely stiffer than the structure. From all selected parameter values, it results a total of 59 different wall cases, considering that a single SC was used when varying the $\gamma_w$ values for a given $N-T$ pair and vice versa.

![Wall cross-sections (WCSs) studied](image)

Figure 3.4 Wall cross-sections (WCSs) studied
3.3. PARAMETRIC STUDY METHODOLOGY

Table 3.2 Varying parameter values for the parametric study

<table>
<thead>
<tr>
<th>N</th>
<th>T</th>
<th>WCS</th>
<th>$A_r$</th>
<th>$\gamma_w$</th>
<th>SC</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0.5</td>
<td>RT</td>
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<td>A, C, D, E</td>
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<tr>
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<td>A, C, D, E</td>
</tr>
<tr>
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<td>4.375</td>
<td>2.0</td>
<td>C</td>
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<td></td>
<td></td>
<td>RT</td>
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<td>C</td>
</tr>
<tr>
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<td>4.375</td>
<td>2.0</td>
<td>C</td>
<td></td>
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<td></td>
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<tr>
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<tr>
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</table>

3.3.2 Seismic design and detailing

For seismic design, each studied wall case was modelled as a fixed-base isolated wall meshed with linear beam elements. For a given $T$ value, the total mass of the system was calculated using the Rayleigh period formula for an uniformly laterally loaded cantilever wall with considerations for shear deformation and for the mass idealization difference between the uniformly distributed mass assumed by the formula and the lumped mass adopted for modeling. 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 $R_d R_o$ (NBCC building importance factor, $I_E = 1$). In other words, the NBCC limitations
about \(V_d\) were omitted as well as the NBCC requirements about accidental torsion. In all cases, the first five lateral mode responses were superposed with the SRSS (Square Root of the Sum of the Squares) method, ensuring a participation of at least 90% of the total mass. Concrete cracking was accounted for by using the effective section properties recommended by CSA standard A23.3-04, assuming an axial compressive force at the wall base, \(P_b\), of \(0.1f'_cA_g\), where \(f'_c\) is the specified concrete compressive strength and \(A_g\) is the gross cross-section area of the wall. The resulting effective properties equal to 70% of the gross properties. The specified material properties used for design are \(f'_c = 30\text{MPa}\) and \(f_y = 400\text{MPa}\) for steel yield strength. Typical wall thicknesses varying between 400 mm and 700 mm were used. The anticipated overall drifts for all studied cases are lower than 1.0%.

A capacity design was performed for each wall case according to CSA standard A23.3-04 to constrain the plastic mechanism at the wall base and prevent shear failure. As required, the capacity design shear envelope, \(V_{cap}\), is the greater of (i) the shear corresponding to the development of the probable moment capacity of the wall base, which is determined as recommended in the Explanatory notes on CSA standard A23.3-04 [CAC, 2006b], that is:

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

(3.9)

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 the specified concrete compressive strength \(f'_c\) and an equivalent steel yield stress of \(1.25f_y\); and (ii) the shear, \(V_{ah}\), corresponding to the development of the factored moment resistance just above the base hinge zone; but is not taken greater than the shear force limit, \(V_{limit}\), which is determined from the elastic shear forces obtained from the modal superposition and reduced with \(R_dR_o = 1.3\). Note that, above the hinge zone, \(V_{ah}\) was generally slightly larger than \(V_p\), but never by more than 10%. Also, as expected, for all wall cases with \(\gamma_w = 4.0\), \(V_{limit}\) controls the design and is less than \(V_p\) by no more than 10%. See Boivin and Paultre, the companion paper, for more details on the required capacity design envelopes. The walls were reinforced in accordance with CSA standard A23.3-04, which means that reinforcement was set within the required minimum and maximum reinforcement limits. Moreover, curtailments of vertical reinforcement along the wall height were such that the wall flexural strength reduction between two adjacent storeys did not exceed between 20% and 10% for short- and
long-period walls, respectively. Preliminary analyses showed that, in some cases, larger strength reductions could produce at the upper storeys, as \( T \) increased, an unintended plastic hinge at the storey just above the reinforcement curtailment even if the flexural strength was greater than the capacity design envelope. Each wall was axially loaded by a static compression force reducing linearly from the base to the top, with a base axial force \( P_b = 0.1f_cA_g \). It is important to note that variations in the material properties or in the base axial compression force have negligible effects on the wall response in comparison with variations in the input earthquake. Therefore, \( f_y \) and \( P_b \) values were sometimes reasonably modified from their nominal value to meet the selected \( \gamma_w \) values. To that end, vertical reinforcement was also set slightly outside the prescribed reinforcement limits in some cases.

### 3.3.3 Modeling for inelastic seismic analysis

The simulation of the inelastic seismic response of each studied wall case was performed with the ITHA using the constant acceleration Newmark method for time integration. Each wall case was modelled as a fixed-base isolated cantilever wall. This modelling assumes that (i) the foundation moment resistance is such that the plastic mechanism forms in the wall only, (ii) there is no rocking of the foundation, and (iii) the soil-structure interaction can be neglected because the soil is largely stiffer than the structure. Note that foundation rocking is now allowed by the NBCC since the 2005 edition. This can significantly reduce the seismic force demand on a wall [Filiatrault et al., 1992].

#### Structural models

Two 2D modelling approaches were adopted for simulating the inelastic seismic response of each studied wall case. For the first approach, the finite element analysis program VecTor2 (VT2) [Wong and Vecchio, 2002] was used. This specialized program enables to simulate at global and local levels the nonlinear static and dynamic behaviors of RC structures from 4-node, smeared material-based membrane elements formulated from the modified compression field theory and the disturbed stress field model [Vecchio, 2000]. With this formulation, reinforcement is assigned as a property to the membrane element and then smeared with concrete properties. This element formulation enables to account for inelastic shear deformation and shear-flexure-axial force interaction. The VT2 constitutive laws selected to model the material responses of concrete and reinforcement steel are given in Table 3.3. The material properties were used at their specified value (nominal). The default laws of all other material responses modelled in VT2 were used for analysis as well as the default values of analysis parameters. Also P-delta effects were taken into account.
The mass storey used for seismic design was lumped at each floor level, as illustrated in Fig 3.5. The slab membrane effect at each floor level was modelled with linear elastic bar elements, as shown in Fig 3.5, assuming an effective width of 1 m and a thickness of 200 mm. Other works have shown the reliability of VT2 to adequately simulate, with very similar modeling considerations and within the limits of similar design ductilities, cyclic and dynamic test responses of RC wall specimens designed with CSA standard A23.3-04 [Ghorbanirenani, 2010]. For the second modeling approach, the open-source software framework OpenSees (OS) (version 2.1.0) [Mazzoni et al., 2006] was used. In this approach, the wall structure was modelled using force-based multilayer beam-column elements with a mesh of one element per storey, and the mass storey was lumped at each floor level, as shown in Fig. 3.5. The hysteretic responses of concrete and reinforcement steel were modelled with the OS uniaxial material laws Concrete03 and Steel02, respectively. The strain hardening and the Bauschinger effect of reinforcement steel were taken into account. The backbone curves in compression and tension of the concrete law were represented with the modified Park-Kent model to account for confinement effects and the modified Bentz model [Vecchio, 2000] to account for tension stiffening, respectively. The material properties were the same as those used for VT2. The wall shear deformation was modelled linearly elastic considering the effective shear area of the wall cross-section. This shear model was aggregated to the element formulation. It results a shear deformation uncoupled from flexural and axial deformations. This modeling is a common simplification and its selection aims to assess the overestimation level it produces on shear response of ductile walls. P-delta effects were modelled with a corotational transformation. The number of integration point (NIP) for each element was initially set to 5 for accuracy but the in-house Tcl (Tool command language) program developed for parametric analysis with OS automatically reduced gradually the NIP up to 3 if convergence failed. This Tcl program also automatically changed within the analysis the nonlinear solution algorithm if convergence failure occurred. Preliminary analyses showed very good agreements between the dynamic deformation and force responses obtained from the VT2 simulations and those obtained from the OS simulations, apart from the OS peak responses generally larger.

Damping model

The damping model used for ITHA with VT2 and OS is the initial stiffness-based Rayleigh damping because this is the sole damping model implemented in VT2. In order to avoid possible problems of spurious damping forces, and hence of force equilibrium, due to high damping in the high modes resulting from this model [Crisp, 1980], Rayleigh damping was specified at the first mode and at the mode number equals to \( N \), which is the last mode,
3.3. PARAMETRIC STUDY METHODOLOGY

Table 3.3 Selected VT2 concrete and steel constitutive laws

<table>
<thead>
<tr>
<th>Material</th>
<th>Constitutive law</th>
<th>Modelled response</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>Popovics (NSC)</td>
<td>compression pre-peak</td>
</tr>
<tr>
<td></td>
<td>Modified Park-Kent</td>
<td>compression post-peak</td>
</tr>
<tr>
<td></td>
<td>Kupfer/Richart</td>
<td>confined strength</td>
</tr>
<tr>
<td></td>
<td>Palermo</td>
<td>hysteretic response with strength decay</td>
</tr>
<tr>
<td></td>
<td>Modified Bentz</td>
<td>tension stiffening</td>
</tr>
<tr>
<td>Steel</td>
<td>Seckin (trilinear)</td>
<td>hysteretic response with Bauschinger effect</td>
</tr>
</tbody>
</table>

NSC: normal strength concrete

Figure 3.5 Structural wall models for ITHA with OpenSees and VecTor2

ensuring that the highest modes of the structure remain sub-critically damped throughout the response. For seismic analysis of multi-degree-of-freedom building structures with $T > 0.5\, \text{s}$, Léger and Dussault [1992] recommended Rayleigh damping 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 $T > 1.5\, \text{s}$. This suggests that
the Rayleigh damping model used in this project does not limit the reach of the obtained results since the selected $T$ values range from 0.5 s to 4.0 s. As damping is intended to model the intrinsic damping of buildings prior to concrete cracking, a modal damping ratio of 2% of critical was assigned to the first and last modes. This modal damping value is a typical mean value for multistorey RC wall buildings, though intrinsic damping of buildings is highly scattered [CTBUH, 2008; Gilles, 2010].

**Input earthquakes**

Atkinson [2009] generated synthetic earthquake time histories that may be used to match the 2005 NBCC uniform hazard spectra (UHS) for eastern and western Canadian regions and for site classes A, C, D and E. These UHS correspond to a 2500-year return period earthquake event. For the western Canadian region and a given site class, Atkinson simulated 45 statistically independent records for each of the following magnitude-distance scenarios associated to crustal and in-slab earthquake events: M6.5 at 10-15 km, M6.5 at 20-30 km, M7.5 at 15-25 km and M7.5 at 50-100 km. M6.5 and M7.5 records correspond to short- and long-duration events, respectively. Since the 2005 and 2010 NBCC UHS for Vancouver are the same, 10 of these records were selected for each scenario of a site class and matched, as recommended by Atkinson, to the design spectra, resulting in 40 UHS-compatible records per site class. Note that the records were matched over period ranges specific to scenarios and not over the whole period range of a spectrum. Each record was used as a horizontal seismic excitation only. The constant time step of the records is either 0.002 s or 0.005 s.

**Inelastic time-history analysis**

The selected 59 wall cases, 40 records per case and 2 modeling approaches result in a total of 4720 analyses. Because of this considerable analysis number, large analysis runtime and the large amount of output data generated by VT2, the analyses were carried out with the 576-node parallel supercomputer of the University of Sherbrooke, producing more than 6 Tb of output data. In order to process this data, MATLAB stand-alone programs were developed and automatically executed after each analysis through a procedure script.

### 3.4 Dynamic analysis results

All predicted demands for a given wall case presented in this section are the means obtained from 40 ground motions. The dynamic shear amplification at a given storey is calculated as the ratio of the mean predicted storey shear force demand to $V_p$ (Eq. (3.9)). The dynamic
shear amplification is calculated at the wall base and as the average value over all storeys (AOS). In addition to these response parameters, the predicted moment, storey shear force and curvature ductility ($\mu_0$) demands over the wall height are presented. The moment and storey shear force demands are normalized relative to the nominal base moment resistance ($M_{n\text{ base}}$) and the predicted base shear force demand ($V_{\text{base}}$), respectively. It is noted that curvature ductility is only predicted with OS by estimating the yield curvature of a wall section from a trilinear 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. Nevertheless, the OS curvature predictions are reasonable estimates because OS generally predicted slightly more flexural yielding than VT2. Note that it was observed that the initiation of a plastic hinge mechanism is associated to a rapid increase of the curvature ductility in the plastic regime and that this transition occurs when $\mu_0 \approx 2$. This value is dependent of the moment-curvature idealization used to determine the yield curvature of a wall section. Also note that the largest mean predicted overall drift for a 5-storey wall is about 1.5% and reduces to about 0.6% for a 40-storey wall.

### 3.4.1 Influence of wall aspect ratio ($A_r$)

Figure 3.6 shows the influence of $A_r$ on dynamic shear amplification and curvature ductility demand for a 10-storey wall with a site class C and $\gamma_w = 2.0$. Based on the OS predictions (Fig. 3.6a), dynamic shear amplification increases with $A_r$, especially for $T = 1.0$ s. The VT2 predictions (Fig. 3.6b), however, indicate no such significant increase for $T = 1.0$ s, even no increase for $T = 1.5$ s, and much lower amplification values. Although not shown, the profiles of the force demands along the wall height predicted with OS and VT2 are quite similar and do not significantly change with the selected $A_r$. As shown in Fig. 3.6c, the main influence of $A_r$ is on the base curvature ductility demand, which largely increases with decreasing $A_r$. For a given $A_r$ value, this ductility demand also increases with $T$. Note that the predicted plasticity height at the wall base, which is the height from the base over which $\mu_0 \geq 1$, is about 10% the wall height ($H$) irrespective of $A_r$. This suggests that $A_r$, and hence the wall length ($l_w$), has a negligible influence on the plasticity height since the wall height is kept constant. This result, however, has to be balanced with the fact that the plasticity height predictions do not account for inelastic shear deformation and shear cracking which can produce significantly larger plasticity heights for walls with low $A_r$ values [Bohl and Adebar, 2011]. The results shown in Figs. 3.6b and 3.6c suggest that there is no relation between the dynamic shear amplification and the curvature ductility
demand at the wall base. Furthermore, in general no plastic hinge mechanism is predicted at the upper storeys in spite of light flexural yielding, as illustrated in Fig. 3.6c.

![Figure 3.6 Influence of wall aspect ratio on seismic response: (a) dynamic shear amplification (from OS); (b) dynamic shear amplification (from VT2); (c) curvature ductility demand (from OS; $\mu_\phi = 1 \equiv$ sectional yielding)](image)

### 3.4.2 Influence of site class (SC)

Figure 3.7 shows the influence of the SC, predicted with VT2, on dynamic shear amplification and shear force demand for wall cases with $N = 5$ and 10 and $\gamma_w = 2.0$. Figures 3.7a and 3.7b show that the selected SC has no significant influence on the AOS dynamic shear amplification, especially for $T \geq 1.0s$. The dynamic shear amplification at the wall base appears to be more sensitive to the selected SC, primarily to SCs A and E likely because of their significantly different respective spectrum, as shown in Fig. 3.2. Although not
shown, the OS predictions suggest a much larger sensitivity of dynamic shear amplification to SC. Note that the force demand profiles along the wall height predicted with OS and VT2 are quite similar and do not significantly change with the selected SC, except for the shear force demand predicted with VT2 for $T = 0.5$ s, as illustrated in Fig. 3.7c. Also, in general no plastic hinge mechanism is predicted at the upper storeys for any SC.

![Graphs showing dynamic shear amplification and normalized storey shear force demand](image)

Figure 3.7 Influence of site class on seismic response (from VT2): (a) dynamic shear amplification for $N = 5$; (b) dynamic shear amplification for $N = 10$; (c) normalized storey shear force demand for $N = 5$

### 3.4.3 Influence of wall cross-section (WCS)

Figure 3.8 shows the influence of the WCS, predicted with OS and VT2, on dynamic shear amplification for a 20-storey wall with a SC D and $\gamma_w = 2.0$. It is observed that the OS predictions differ significantly in magnitude and trend from the VT2 predictions. The OS predictions suggest that the WCS has no influence on dynamic shear amplification.
whereas the VT2 predictions indicate a linear decrease of amplification between the WCSs RT and H1, resulting in a base amplification reduction of about 20% between these two WCSs. The dynamic shear amplifications at the wall base predicted with OS for the WCSs RT and H1 are 10% and 40% larger than those predicted with VT2, respectively. This brings up the importance of accounting for nonlinear shear deformation when predicting the seismic shear response of flanged walls. Note that the force demand profiles along the wall height predicted with OS and VT2 are almost the same and do not significantly change with the selected WCS. Also in general no plastic hinge mechanism is predicted at the upper storeys for any WCS.

![Figure 3.8 Influence of wall cross-section on dynamic shear amplification (from OS and VT2)](#)

### 3.4.4 Influence of wall base overstrength ($\gamma_w$)

Figure 3.9 shows the influence of $\gamma_w$, predicted with VT2, on dynamic shear amplification and force demand for wall cases with $N = 5, 10, 15, 20$ and $40$. The predictions show that dynamic shear amplification rapidly decreases, almost linearly sometimes, with increasing $\gamma_w$, for any $N$ value. As the $\gamma_w$ values increase from 1.3 to 4.0, the mean base shear amplification values predicted for all $N$ values decrease from a maximum of 2.7 to a minimum of 1.0. For comparison purposes, the corresponding values predicted with OS are 3.1 and 1.2. The maximum amplifications values, which are associated to $T = 4.0$ s and $\gamma_w = 1.3$, predicted with VT2 and OS are by far much lower than the amplification values calculated with $\omega^*_{i}$ (Eq. (3.4)) and $\varepsilon^*$ (Eq. (3.7)) for the same $T - \mu_{\Delta - \gamma_w}$ values, as observed in Fig. 3.1. Figure 3.10, which compares all dynamic shear amplification predictions given in Fig. 3.9, shows that, for $N = 5$, dynamic shear amplification significantly increases with increasing $T$ from 0.5 s to 1.0 s, and, for the other $N$ values, dynamic shear amplification
increases slowly, or reduces sometimes, with increasing \( N \) and \( T \). Also this figure shows that \( \omega_v \) (Eq. (3.1)) conservatively estimates dynamic shear amplification only for \( \gamma_w \geq 3.0 \). For \( \gamma_w = 1.3 \), the reductions of base shear amplification observed in Fig. 3.10 as \( T \) increases for a given \( N \) result of significant flexural yielding (\( \mu_\phi > 2 \)) at the upper storeys, as illustrated in Fig. 3.11. Although \( \gamma_w \) affects dynamic shear amplification, Fig. 3.9 indicates that \( \gamma_w \) has no significant influence on the storey shear force profile along the wall height for \( T \geq 1.0 \), resulting in very similar profiles.

For the flexural demand, Fig. 3.9 shows that \( T \) and \( \gamma_w \) have a significant influence. Actually as \( T \) increases so does the flexural demand, particularly at the upper storeys, but as \( \gamma_w \) increases, this demand reduces rapidly without, however, inhibiting the plastic hinge mechanism at the wall base, even for \( \gamma_w = 4.0 \), as illustrated in Fig. 3.11. Curiously Fig. 3.11 shows that the curvature ductility demand at the upper storeys for \( \gamma_w = 1.3 \) and a given \( T \) reduces with increasing \( N \). Actually, this reduction results from the required minimum flexural reinforcement which produces at these storeys, as \( N \) increases, moment resistances increasingly larger than the capacity design moment envelope. Despite this large overstrength, flexural yielding is predicted at the upper storeys, meaning that the flexural demand has significantly exceeded the capacity design envelope. In general, however, no such excess is predicted for \( T = 0.5 \) s and for \( \gamma_w \geq 3.0 \) irrespective of \( T \). Figure 3.11 shows that in general no plastic hinge mechanism is predicted at the upper storeys (\( \mu_\phi < 2 \)) for \( \gamma_w \geq 2.0 \) despite sometimes light flexural yielding. Figure 3.9 shows 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 \( \gamma_w \) increases. Similar observations can be made for curvature ductility demand (see Fig. 3.11d). 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 \( \gamma_w \) increases.

From Fig. 3.11, note that, for \( \gamma_w \geq 2.0 \), the plasticity height at the wall base decreases, with respect to \( H \), from about 20% to 2.5% of \( H \) as \( N \) (or \( H \)) and \( \gamma_w \) increase. This differs from the relation \( 0.5l_w + 0.1H \) prescribed by CSA standard A23.3-04 for determining the base plasticity height requiring special detailing, where the estimated plasticity height is at least 10% of \( H \) and independent of \( \gamma_w \). This shows that this relation is inadequate, giving too conservative base plasticity height estimates for tall walls with large flexural overstrength at the wall base. Although the plasticity height predictions does not account for inelastic shear deformation and shear cracking, their influence should reduce with increasing \( H \) and \( \gamma_w \), and hence should not change the previous result.
CHAPTER 3. PARAMETRIC STUDY OF DUCTILE CANTILEVER WALLS

Figure 3.9 Influence of wall base overstrength ($\gamma_w$) on dynamic shear amplification and shear force and moment demands: (a) $N = 5$; (b) $N = 10$; (c) $N = 15$; (d) $N = 20$ and 40
3.5 Discussion

The results presented in the previous sections show that, from the studied parameters, those affecting the most dynamic shear amplification and seismic force demand in ductile RC walls are $T$ and $\gamma_w$. Actually they show that, for a given $T$, the relative higher mode contribution in the seismic force demand highly depends on $\gamma_w$. While for any $T$ dynamic shear amplification largely reduces with increasing $\gamma_w$, $\gamma_w$ has no significant influence on the shear force demand profile for $T \geq 1.0s$. Moreover, for $T > 1.0s$, the dynamic shear amplification values slightly increase for $\gamma_w < 2.0$ and remain almost constant for $\gamma_w \geq 3.0$, as $T$ increases (see Fig. 3.10). In general the mean AOS and base shear amplifications values predicted with VT2 are much larger than 1.0, with a maximum of 2.7, meaning that the predicted shear force demands have significantly exceeded $V_p$ (Eq. (3.9)), which is the capacity design shear envelope for the plastic hinge zone of wall cases with $\gamma_w < 4.0$.

This shows one more time that the capacity design methods prescribed by CSA standard A23.3-04 for shear strength design can produce inadequate design envelopes. In spite of the very large exceeding shear forces, the VT2 predictions showed at worst light shear cracking and light onset of shear reinforcement yielding, even if the shear resistance of each wall case was set to match the capacity design envelope. The recent dynamic test results of Ghorbanirenani [2010] of large scale 8-storey MD wall specimens designed according to CSA standard A23.3-04 showed stable hysteretic shear responses, no shear reinforcement yielding and no shear failure of the specimens for base shear demands corresponding to...
(p) \( p = N \) (q) \( N = N \) (r) \( N = N \) (s) \( N = N \) (t) \( N = N \) (u) \( N = N \) (v) \( N = N \) (w) \( N = N \) (x) \( N = N \) (y) \( N = N \) (z) \( N = N \)

Figure 3.11 Influence of wall base overstrength on curvatureductility.

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up to 150% the nominal shear resistance (based on the actual material strengths) of the wall or 200% the design earthquake. These observations suggest that either (i) the shear resistance requirements of CSA standard A23.3-04 for MD and ductile shear walls are highly conservative; (ii) the energy associated to the high peak shear forces, which are generated by higher mode responses, is not sufficient to sustain the displacement necessary for a shear failure because of the transient nature of these forces and of the unlikelihood that these forces occur simultaneously with the peak displacement responses, which are dominated by the first mode response [Lybas, 1981]; or (iii) a combination of both assumptions. The observations made by Ghorbanirenani [2010] support the first assumption by suggesting a significantly higher contribution of concrete in the base hinge region to shear resistance than that required for design. The predictions obtained in the present work agree with the second assumption because they show that the peak base shear forces and the peak top displacements never occur simultaneously and the base shear forces corresponding to the peak top displacements are generally much lower than the maximum base shear force. The previous assumptions need further investigation but this is out of the scope of this paper. Meanwhile, the predictions obtained in this work indicate that none of the dynamic shear amplification factors presented in Section 3.2 can adequately estimate, in their current form, the predicted amplification values because their formulation does not generally account for \( T \) and \( \gamma_w \) and/or is not adapted to Canadian seismic provisions. These issues result in estimates that largely differ from those predicted, as observed from Figs. 3.1 and 3.10. An adequate general formulation should not only account for \( T \) and \( \gamma_w \) but also for the seismic zone and \( \mu_\Delta \) (or \( R_dR_o \)).

The predictions showed that \( T \) and \( \gamma_w \) largely influence the moment demand, which increases with increasing \( T \) and reducing \( \gamma_w \). For \( T \geq 1.0 \) and \( \gamma_w < 3.0 \), the predicted flexural demand at the upper storeys has always significantly exceeded the capacity design moment envelope and this excess reduces with increasing \( \gamma_w \). This shows that the capacity design method prescribed by CSA standard A23.3-04 for flexural strength design can produce inadequate design envelopes. In general, for \( \gamma_w \geq 2.0 \) and any \( T \), the plastic hinge mechanism is constrained at the wall base, as expected, despite sometimes light flexural yielding at the upper storeys (\( \mu_\phi < 2 \)). This suggests that plastic hinge formation at the upper storeys might be precluded if a minimum \( \gamma_w \) value of 2.0 is forced at design stage. An additional hinge inhibition would appear when flexural design above the base hinge zone is governed by the required minimum reinforcement, as observed in Fig. 3.11. It is important to note that these statements only apply to regular wall structures without stiffness and/or strength irregularities. These irregularities are prone to plastic hinge formation. For instance, preliminary analyses with \( \gamma_w = 2.0 \) and \( T \geq 1.0 \) predicted at the
upper storeys a plastic hinge at the storey just above a reinforcement curtailment if the wall flexural strength reduction between these two adjacent storeys exceeded about 20% for wall cases with $T = 1.0$ s and about 10% for cases with $T = 4.0$ s (note that no such sensitivity to strength reductions was observed with $\gamma_w \geq 3.0$). Also the 8-storey moderately ductile RC wall specimens ($\gamma_w = 1.145 \equiv 2.29$ for ductile walls, see below) dynamically tested by Ghorbanirenani [2010] with design-level excitations experienced plastic hinge formation at the symmetric setback located just above the wall mid-height. Therefore, all these results suggest that a SPH design may be inadequate and unsafe for regular ductile cantilever wall structures with $\gamma_w < 2.0$ and for wall structures with stiffness and/or strength irregularities at the upper storeys. A dual-plastic hinge design [Ghorbanirenani, 2010; Panagiotou and Restrepo, 2009] may be a better alternative. An additional plastic hinge mechanism at the upper storeys enables normally to slightly reduce base shear amplification, as shown in Fig. 3.10 for $\gamma_w = 1.3$.

Note that some of the above results may certainly apply to moderately ductile RC walls designed with CSA standard A23.3-04. For these walls, the product $R_dR_o = 2.0 \cdot 1.4 = 2.8$, which is half of the total force reduction factor for ductile walls ($R_dR_o = 5.6$). This means that the results obtained for ductile walls with $\gamma_w \geq 2.0$ might theoretically be extended to MD walls with $\gamma_w \geq 1.0$. Using this assumption, Fig. 3.10 suggests a dynamic base shear amplification value slightly above 1.5 for MD walls with $\gamma_w = 1.3$ ($\equiv 2.6$ for ductile walls). This suggests that CSA standard A23.3-04 should also account for dynamic shear amplification for shear strength design of MD walls, given that the seismic provisions for these walls are much less stringent than for ductile walls. However, the excellent shear performance of the MD wall specimens dynamically tested by Ghorbanirenani [2010] for motion intensities corresponding to up to 200% the design earthquake suggests that it is unnecessary.

### 3.6 Conclusion

In this work, a parametric study of regular ductile RC cantilever walls designed with the 2010 NBCC and CSA standard A23.3-04 for Vancouver was performed in order to investigate the influence of the following parameters on the higher mode amplification effects and hence on the seismic force demand: number of storeys ($N$), fundamental lateral period ($T$), site class (SC), wall aspect ratio ($A_r$), wall cross-section (WCS) and wall base flexural overstrength ($\gamma_w$). The study is based on ITHAs, carried out with a large suite of design-level ground motions, of fixed-base isolated walls modelled with two different 2D modeling approaches: a multilayer beam approach (OpenSees) modeling shear deformation
3.6. CONCLUSION

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. From this study, the following main conclusions can be drawn:

1. Not accounting for inelastic shear deformation and shear-flexure-axial interaction can produce dynamic shear amplification predictions that are much larger in magnitude and inadequate in trend when shear deformation in wall response is significant.

2. The relation $0.5l_w + 0.1H$ prescribed by CSA standard A23.3-04 for determining the base plasticity height requiring special detailing is inadequate, giving too conservative estimates for tall walls with large flexural overstrength at the wall base.

3. The studied parameters affecting the most dynamic shear amplification and seismic force demand are $T$ and $\gamma_w$.

4. While for any $T$ dynamic shear amplification significantly reduces with increasing $\gamma_w$, $\gamma_w$ has no significant influence on the shear force demand profile for $T \geq 1.0\, s$. Moreover, for $T > 1.0\, s$, dynamic shear amplification slightly increases for $\gamma_w \leq 2.0$ and remains almost constant for $\gamma_w \geq 3.0$, as $T$ increases.

5. None of the reviewed dynamic shear amplification factors can adequately estimate, in their current form, the predicted amplification values because their formulation does not generally account for $T$ and $\gamma_w$ and/or is not adapted to Canadian seismic provisions. An adequate general formulation should not only account for $T$ and $\gamma_w$ but also for the seismic zone and $\mu_\Delta$ (or $R_dR_o$).

6. The capacity design methods prescribed by CSA standard A23.3-04 for 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 the wall.

7. A minimum $\gamma_w$ value of 2.0 can generally preclude the unintended hinge formation at the upper storeys and constrain the plastic mechanism at the wall base, as expected. However, for walls with $2.0 \leq \gamma_w < 3.0$, this observation applies if reinforcement curtailment along the wall height does not result in a flexural strength reduction, between two adjacent storeys, exceeding about 20% to 10% for walls with $T_1$ ranging from 1.0\, s to 4.0\, s, respectively.

8. A SPH design may be inadequate and unsafe for regular ductile cantilever wall structures with $\gamma_w < 2.0$ and for wall structures with stiffness and/or strength irregularities at the upper storeys.
As this work is based on the seismic region of Vancouver, some conclusions may not necessarily apply to regions with different seismicity. A similar work is in progress for the eastern Canadian regions.

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CHAPTER 4

New capacity design methods

Foreword

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Titre français: Demande sismique en force sur des murs de refend ductiles en béton armé: partie 2 - nouvelles méthodes de dimensionnement à la capacité

Paper’s contribution to this project: This paper contributes to the thesis first by reviewing various capacity design methods proposed in the current literature and recommended by design codes for determining capacity design moment and shear envelopes for ductile concrete walls, and then by proposing new capacity design methods for the Canadian Standards Association (CSA) standard A23.3 for flexural and shear strength design of regular ductile concrete cantilever walls.
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, effets d'amplification des modes supérieurs, demande sismique en force
4.1 Introduction

In order 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, in the companion paper (referred to herein as only the companion paper) [Chapter 3 of this document], 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.

4.2 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 [ACI, 2008] 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 ($M_f, V_f$) from linear elastic analysis, where $R_d$ and $R_o$ are the ductility- and overstrength-related force reduction factors, respectively. For ductile RC cantilever walls, $R_d = 3.5$ and $R_o = 1.6$, which gives $R_dR_o = 5.6$.

4.2.1 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, $M_n$, 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, $M_r$, 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 $M_n/M_f$ at the wall base and not less than 1.3.

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 $M_r/M_f$ calculated at the top of $h_p$, where $l_w$ and $H$ are the wall length and height, respectively (see Fig. 4.1a). 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. 4.1a. Various works [Boivin, 2006; Tremblay et al., 2001], 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.

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. 4.1b. 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
CHAPTER 4. NEW CAPACITY DESIGN METHODS

Figure 4.1 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].
4.2. REVIEW OF CAPACITY DESIGN METHODS

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.

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. 4.1 c. 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 $2h_s$, 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.

Bachmann and Linde [1995]

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. 4.1 d, their envelope presents a constant strength $R_p$ over an assumed base hinging region of height $l_p$, which can be estimated as the larger of $l_w$ or $H/6$. $R_p$ is set equal to or greater than $\gamma_R M_f$, where $\gamma_R$ is the resistance factor taken as 1.2. As seen from Fig. 4.1 d, an increased strength is suggested immediately above the hinging region in order 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 $\lambda_o R_p$ where $\lambda_o$ 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 $l_{ec} = \alpha_{ec} l_e = 0.20 T_1 l_e$ where $T_1$ is the fundamental lateral period of the wall in the direction.
under consideration. Above the height $l_{ec}$, the linear envelope as proposed by Paulay and Priestley [1992] is considered until a possible minimum flexural strength $R_{min}$ is reached due to nominal minimum reinforcement requirements. According to Bachmann and Linde, $\alpha_{ec} = 0.20 T_1$ 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.

**Priestley and Amaris [2003]**

For walls designed according to the direct displacement-based design (DDBD) method proposed by Priestley and Kowalsky [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 which 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:

$$M_{MMS,i} = 1.1 \times \sqrt{M_{1,i}^2 + M_{2e,i}^2 + M_{3e,i}^2 + \ldots} \quad (4.1)$$

with $M_{1,i} = \min (M_{F,i}; M_{1e,i})$ where $M_{F,i}$ is the ductile design (first-mode) moment at level $i$ determined from DDBD and $M_{1e,i}, M_{2e,i}, M_{3e,i}$ 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. 4.1e. $M_b^o$ is equal to $\phi^o M_{F,base}$ where $\phi^o$ 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.

**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. 4.1f, this version consists in a bilinear capacity envelope defined by $M^o_H$, the mid-height overstrength moment $M^o_{0.5H} = A_T M^o_H$, and zero moment at the wall top, with the moment ratio $A_T$ given by

$$A_T = 0.4 + 0.075 T_1 \left( \frac{\mu_{\Delta}}{\phi^0} - 1 \right) \geq 0.4$$

(4.2)

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 $l_w/2$, as illustrated in Fig. 4.1f. It is interesting to note that Eq. (4.2) is not bounded by an upper limit, meaning that the moment ratio could be equal to or larger than 1, as shown in Fig. 4.2 (with $T = T_1$), resulting in $M^o_{0.5H} \geq M^o_H$. As previously discussed for the design envelope shown in Fig. 4.1d, the feasibility of such design may be simply impossible.

![Figure 4.2](image.png)

**4.2.2 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.

**CSA standard A23.3-04**

As illustrated in Fig. 4.3a, CSA standard A23.3-04 requires that the capacity design shear envelope be the 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, 2006b], that is:

\[
V_p = V_f \left( \frac{M_p}{M_f} \right)_{\text{base}} \tag{4.3}
\]

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

\[
V_{ah} = V_f \left( \frac{M_r}{M_f} \right)_{h_p \text{top}} \tag{4.4}
\]

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 \( \gamma_w \geq 5.6/1.3 \approx 4.3 \). 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 overstrengthed in flexure.

**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, \( V_E^o \), is obtained as follows (see Fig. 4.3b):

\[
V_E^o = \omega_v \phi_o V_E = \omega_v \left( \frac{M_{o,w}}{M_E} \right)_{\text{base}} V_E \tag{4.5}
\]
4.2. REVIEW OF CAPACITY DESIGN METHODS

Figure 4.3  Capacity design shear envelopes: (a) CSA standard A23.3-04; (b) Paulay and Priestley [1992]; (c) Priestley et al. [2007]; (d) 2004 EC8 for ductile wall systems; (e) 2004 EC8 for ductile wall-frame systems; (f) Rutenberg and Nsieri [2006].
where $V_E$ and $M_E$ are the design shear force and bending moment diagrams derived from the code-specified inverted triangular force distribution, respectively, $\phi_o$ is the flexural overstrength factor at the wall base and is calculated as the ratio of the wall base moment capacity, $M_{o,w}$, 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

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

(4.6)

where $N$ is the number of storeys of the building. Equation (4.6) is based on the work of Blakeley et al. [1975]. Paulay and Priestley limit $V_E^0$ to $\mu_\Delta V_E$, 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. (4.5) can be very unconservative because of the significant underestimation of the higher mode shear amplification by $\omega_v$.

**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:

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

(4.7)

with $V_{1,i} = \min (V_{F,i}; V_{1e,i})$ where $V_{F,i}$ is the ductile first-mode shear force at level $i$ determined from DDBD and $V_{1e,i}, V_{2e,i}, V_{3e,i}$ 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.

**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. 4.3c. The capacity design base shear force, $V_b^0$, is equal to $\omega_v^* \phi_o V_{F,base}$ with
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\[
\omega^*_\nu = 1 + B_T \mu_\Delta / \phi_o \quad (4.8)
\]

\[
B_T = 0.067 + 0.4 (T_1 - 0.5) \leq 1.15 \quad (T_1 \geq 0.5 \text{s}) \quad (4.9)
\]

where \( \omega^*_\nu \) is a dynamic shear amplification factor and \( \phi_o \) is taken as \( M_o.w / M_F \) at the wall base. The design shear force at the top of the wall, \( V_n^o \), is equal to \( C_T V_b^o \) with

\[
C_T = 0.9 - 0.3 T_1 \geq 0.3 \quad (4.10)
\]

2004 Eurocode 8

As illustrated in Fig. 4.3d, 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 \( (q < 3) \) and, for highly ductile (HD) walls \( (q > 3) \), calculated from Eq. (4.11), which is based on the formula proposed by Keintzel [1990] to amplify the seismic shear forces obtained from the code-specified equivalent static analysis:

\[
\varepsilon = q \cdot \sqrt{\left( \frac{\gamma_{Rd}}{q} \cdot \frac{M_{Rd}}{M_{Ed}} \right)^2 + 0.1 \left( \frac{S_e(T_c)}{S_e(T)} \right)^2} \quad (4.11)
\]

with \( 1.5 \leq \varepsilon \leq q \) where \( q \) is the behavior factor (equivalent to \( \mu_\Delta \)) used for design, \( M_{Rd} \) is the design flexural resistance at the wall base, \( M_{Ed} \) is the design bending moment (equivalent to \( M_f \)) at the wall base, \( \gamma_{Rd} \) is the factor to account for overstrength due to steel strain-hardening (may be taken as 1.2), \( T_c \) 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. 4.3e where \( V_{Edb} \) is the capacity design shear force at the wall base. Note that the design envelope shown in Fig. 4.3d may have a similar profile to that shown in Fig. 4.3a if \( V_f \) is determined from dynamic analysis. Various works [Priestley and Amaris, 2003; Rutenberg and Nsieri, 2006] showed that Eq. (4.11) 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.
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. 4.3 where the capacity design shear force at the wall base, $V_{db}$, is taken as

$$V_{db} = \varepsilon^* V_{by} = [0.75 + 0.22(T_1 + q + T_1 q)] V_{by} \quad (4.12)$$

$$V_{by} = \frac{M_{by}}{\frac{2}{3} H (1 + \frac{1}{2N})} \quad (4.13)$$

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

$$\xi = 1.0 - 0.3 T_1 \geq 0.5 \quad (4.14)$$

Each one of the expressions $0.1 H$ and $\xi H$ in Fig. 4.3 should be taken as an integer number of storeys. Equation (4.12) 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. (4.14) 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$(l_{w1} H/6)$ and the capacity design base shear force is equal to $\beta^b V_f$ with the dynamic shear amplification factor $\beta^b$ given by

$$\beta^b = 1.0 + (0.281 T_1 + 0.394)((R/\psi^o) - 1.5)^{0.553} \quad (4.15)$$

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 $M_r/M_f$ at the wall base.
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4.2.3 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.

4.3 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.

4.3.1 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 $M_r \geq M_f$ and then, for capacity
design considerations, that the wall sections above the base hinging zone $h_p$ be designed such that $M_r \geq M_f$ amplified by the ratio $M_r/M_f$ calculated at the top of the hinging zone, as shown in Fig. 4.1a. 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 limit the unintended plastic action above the base hinging 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. 4.1f.

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. 4.1f. 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, $M_0^c$, and the moment ratio $A_T$ (Eq. (4.2)) of mid-height overstrength moment, $M_{0.5H}$, to $M_0^c$. Note that $M_0^c$ 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.

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

$$M_r \geq M_{bo} = \phi_s \gamma_w^{\min} M_f = \gamma_b M_f$$

(4.16)

where $M_{bo}$ 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 $\gamma_b$ is the minimum factored base overstrength and is equal to 1.7. Note that $\gamma_w^{\min} = 2.0$ 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, $\gamma_b$ would be equal to $1.7/1.05 \approx 1.6$. Although the requirement given by Eq. (4.16) 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 new 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 4.4 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. 4.1b and c corresponds to a moment ratio of 0.50, lower moment ratio values are irrelevant for design purposes. Figure 4.4 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 4.1 gives the proposed $\alpha_M$ values for determining the mid-height moment, $M_{0.5H}$, of the bilinear envelope. In Table 4.1, the effective force reduction factor $R_d R_o / \gamma_w$ is used instead of solely $\gamma_w$ in order 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, $R_d R_o = 5.6$. Note that the moment ratio values shown in Fig. 4.4 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 4.1 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 bilinear envelope to account for tension shift. Therefore, it is proposed that the capacity design base moment, $M_{n_b}$, be determined for a nominal strength based on the specified material strengths, as specified by CSA standard A23.3-04.

![Figure 4.4](image)

Figure 4.4 Predicted maximum moment ratios vs fundamental lateral period for wall overstrength factor ($\gamma_w$) values equal to 2.0, 3.0 and 4.0 (data from Boivin and Paultre, companion paper).

<table>
<thead>
<tr>
<th>$R_d R_o / \gamma_w$</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>
As shown in Fig. 4.1f, Priestley et al. [2007] proposed a vertical shift of 0.5\(l_w\) 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 0.5\(l_w\) corresponds to the case where the entire shear is resisted solely by shear reinforcement, neglecting concrete contribution, and a shift of \(l_w\), 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 0.5\(l_w\) 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 0.5\(l_w\) + 0.1\(H\), 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. 4.5a 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. 4.5b. 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 \(l_w\), 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. 4.5b, the constant terms of the linear trend lines are lower than \(h_s\). In order 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. 4.5c. From these predictions, the following relation for estimating \(h_p\) is proposed for design purposes:

\[
h_p = 0.8h_s + \beta_0 H \geq \max(h_s; 0.5l_w)\tag{4.17}
\]
Figure 4.5 Predicted base plasticity heights and associated trend lines for wall overstrength factor ($\gamma_w$) values equal to 2.0, 3.0 and 4.0: (a) mean plasticity height normalized to wall height; (b) mean plasticity height; (c) M–SD plasticity height (data from Boivin and Paultre, companion paper).
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where \( \beta_0 \) is equal to 0.10, 0.05 and 0.03 for \( R_dR_o/\gamma_w \) 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. (4.17) 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. (4.17) slightly differs from that of the common relation for walls, that is, \( h_p = \alpha l_w + \beta H \) where \( \alpha \) and \( \beta \) are constants. This difference may found an explanation in the fact that the common relation is based on plasticity lengths that were measured from RC beam tests and tests of RC walls laterally loaded at their top [Bohl and Adebar, 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. (4.17) requires a special ductile detailing over the entire wall height. This is too conservative. The code-specified relation \( h_p = 0.5l_w + 0.1H \), which would be an upper bound for such walls [Bohl and Adebar, 2011], is more appropriate for this particular case, though the relation \( 0.5l_w + \beta dH \) accounting for the base flexural overstrength might be a better alternative. This should be investigated because it could result in wall designs with less special ductile detailing, and hence more economical, for walls with large \( \gamma_w \) values.

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. 4.6a. This envelope is obtained as follows:

1. Determine the minimum base overstrength moment \( M_{bo} \) by scaling up the factored design base moment \( M_{f\text{base}} \) with the minimum factored base overstrength \( \gamma_b = 1.7 \);
2. Determine the required flexural reinforcement content at the wall base to satisfy both Eq. (4.16) 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 \( \gamma_w = M_{nb}/M_{f\text{base}} \), and then \( R_dR_o/\gamma_w \);
5. Using \( R_dR_o/\gamma_w \) and the fundamental lateral period \( T_1 \) of the wall system, determine from Table 4.1 the moment ratio \( \alpha_M \) and then calculate the mid-height moment \( M_{0.5H} = \alpha_M M_{nb} \). Linear interpolation on \( R_dR_o/\gamma_w \) and \( T_1 \) may be used to get \( \alpha_M \);
6. From \( M_{nb} \) and \( M_{0.5H} \), draw the bilinear envelope as illustrated in Fig. 4.6a;
7. Determine the plastic hinge height \( h_p \) using Eq. (4.17). This height should be taken as an integer number of storeys. Linear interpolation on \( R_dR_o/\gamma_w \) may be used;
8. Vertically shift first the whole bilinear envelope of 0.5$l_w$ and then the base vertical line up to $h_p$.

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 $T$ ranging from 1.0 s to 4.0 s, respectively.

4.3.2 Shear strength design

As mentioned in Section 4.2.2, 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. (4.3)) and $V_{ah}$ (Eq. (4.4)), 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. 4.3f. As shown in this figure, four parameters define the design envelope of Rutenberg and Nsieri: the capacity design base shear force $V_{db}$ (Eq. (4.12)), 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. (4.14)) 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:

$$V_{pb} = \bar{\nu}V_{p \text{ base}} \leq V_{\text{limit base}}$$  \hspace{1cm} (4.18)

where $\bar{\nu}$ is a dynamic shear amplification factor, $V_{p \text{ base}}$ is the probable shear force $V_p$ (Eq. (4.3)) at the wall base and $V_{\text{limit base}}$ 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. The derivation of $\bar{\nu}$ is based on the dynamic shear amplification results presented in the companion paper for different wall cases characterized, among others, by the number of storeys ($N$), the fundamental lateral period ($T_1$) and the base flexural overstrength.
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Figure 4.6 Proposed capacity design envelopes: (a) flexural strength design; (b) shear strength design

$(\gamma_w)$ of the wall structure. These results are summarized in Fig. 4.7, where 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 4.7 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 $\overline{\varphi}_\nu$ values for design purposes, the following discussion needs to be addressed. In order to be conservative, it would be reasonable to set the proposed $\overline{\varphi}_\nu$ values larger than the mean base predictions shown in Fig. 4.7 or even better, to set them from the $M-SD$ base predictions, which are in general about 15% larger than the mean base predictions. Such conservatism, however, is judged unnecessary because of the 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 to up to 150% the nominal shear resistance (based on the actual material strengths) of the wall or 200% the design earthquake [Ghorbanirenani, 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.

Based on the previous 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 the $\overline{\varphi}_\nu$ 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. 4.7. The $\overline{\varphi}_\nu$ 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 $V_{\text{limit}}$. Since life safety is a priority, Table 4.2 gives the proposed $\overline{\varphi}_\nu$ 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 adequate and conservative for multistorey walls. Thus, this parameter is taken as $0.5V_{pb}$ for
4.3. PROPOSED CAPACITY DESIGN METHODS

The height ratio \( \xi \), given by Eq. (4.14) and shown in Fig. 4.3f, 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 \( T_1 = 0.5 \text{s} \) and \( T_1 \geq 1.0 \text{s} \), respectively. From these observations, a new height ratio \( \bar{\xi} \) is proposed:

\[
0.5 \leq \bar{\xi} = 1.5 - T_1 \leq 1.0
\]  

(4.19)

As shown in Fig. 4.3f, there is a height of 0.1\( H \) 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. (4.17).
Figure 4.66 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. (4.3);
2. Determine the dynamic shear amplification factor \( \bar{\omega}_v \) from Table 4.2 using \( R_d R_o/\gamma_w \) and the fundamental lateral period \( T_1 \) of the wall system. Linear interpolation on \( R_d R_o/\gamma_w \) and \( T_1 \) may be used.
3. Calculate the capacity design base shear force \( V_{pb} \) with Eq. (4.18) and the capacity design top shear force as \( 0.5V_{pb} \);
4. Calculate the height \( \bar{\xi} H \) using Eq. (4.19) for \( \bar{\xi} \). This height should be taken as an integer number of storeys;
5. Determine the plastic hinge height \( h_p \) with Eq. (4.17);
6. Draw the capacity design envelope as shown in Fig. 4.66.

### 4.4 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 multistorey 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\% [Gilles, 2010; Porter et al., 2002], in modal damping ratios measured in actual multistorey 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 \( V_p \), is about 1.5 for the isolated cantilever wall system. From the previous \( T_1 \) and \( \gamma_w \) values, Tables 4.1 and 4.2 give by linear interpolation a moment ratio \( \alpha_M \) of about 0.52 and a dynamic shear amplification factor \( \overline{\omega}_v \) of about 1.17, respectively. These values are lower than the previous ones, meaning that the proposed \( \alpha_M \) and \( \overline{\omega}_v \) values can be unconservative for eastern regions having a seismic hazard similar to that of Montreal. 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 way 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 \( \omega_v \) (Eq. (4.6)) to calculate a reduced dynamic shear amplification factor for estimating the wall base shear force for capacity design. By replacing \( \omega_v \) by \( \bar{\omega}_v \) (Table 4.2) in this relation, the following reduced amplification factor \( \bar{\omega}_v^* \) may be used to calculate \( V_{ph} \) (Eq. (4.18)) for walls that are part of wall-frame systems:

\[
\bar{\omega}_v^* = 1 + (\bar{\omega}_v - 1)\eta
\]  

(4.20)

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 (\( R_d R_o = 2.0 \cdot 1.4 = 2.8 \)), 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 $\gamma_w = 1.3$, $R_d R_o / \gamma_w = 2.15$ and Table 4.1 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. (4.16)) does not apply to MD walls because the possible $R_d R_o / \gamma_w$ 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 4.2 gives a $\bar{\omega}_o$ 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 to up to 200% the design earthquake suggests, however, that a $\bar{\omega}_o$ value of 1.0 would be sufficient for MD walls. This should be further investigated.

4.5 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 multistorey 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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CHAPTER 5

Conclusion and Future Research

5.1 Conclusion

This research project proposes for CSA standard A23.3 new capacity design methods, considering higher mode amplification effects, for determining, for a single plastic-hinge (SPH) design, capacity design envelopes for flexural and shear strength design of regular ductile RC cantilever wall structures used as seismic force resisting system (SFRS) for multistorey buildings. In order to achieve this objective, first the seismic performance of a 12-storey ductile RC core wall system designed according to the 2005 NBCC and the 2004 CSA standard A23.3 is assessed. This study aimed to assess from a realistic case under design-level ground motions and above the adequacy of the capacity design methods prescribed by the CSA standard A23.3-04 for flexural and shear strength design of ductile RC shear walls. Second, an extensive parametric study is conducted to investigate the influence of various parameters on the higher mode amplification effects, and hence on the seismic force demand, in regular ductile RC cantilever walls designed with the 2010 NBCC and the 2004 CSA standard A23.3. The parametric study is based on inelastic time-history analyses of fixed-base isolated cantilever walls performed with a common multilayer beam approach and a smeared membrane approach accounting for inelastic shear-flexure-axial interaction and deformation according to the modified compression field theory (MCFT) and the disturbed stress field model [Vecchio, 2000]. This study aimed to determine the parameters affecting the most the seismic force demand on these walls under design-level ground motions and to assess in a more general way the adequacy of the capacity design methods prescribed by the CSA standard A23.3-04 for flexural and shear strength design of such walls. Third, a review of 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 walls is performed. This review aimed to bring out the limitations of the current methods in estimating the seismic force demand on ductile walls whose seismic force response is governed by higher lateral mode responses. From the outcomes of this review and the parametric study, new capacity design methods are proposed and a discussion on the limitations of these methods and on their applicability to various wall systems is discussed.
From the work performed in this project, the following main conclusions can be drawn, with regard to:

**Seismic design of ductile RC walls**

1. The 2005/2010 NBCC spectral accelerations ($S_a(T)$) underestimate the elastic responses of higher lateral modes of multistorey walls because their traditional 5% damping overestimates actual damping (about 2% on average) and hence reduces the higher mode responses.

2. The capacity design methods prescribed by CSA standard A23.3-04 for 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 the wall. This results from a deficiency of the capacity design methods to not take into account the higher mode amplification effects in the inelastic regime.

3. The relation $0.5\ell_w + 0.1H$ prescribed by CSA standard A23.3-04 for determining the base plasticity height requiring special ductile detailing is inadequate, giving too conservative estimates for tall walls with large flexural overstrength at the wall base.

4. A SPH design may be inadequate and unsafe for regular ductile cantilever wall structures with a wall overstrength factor ($\gamma_w$) less than 2.0 and for walls with stiffness and/or strength irregularities at the upper storeys.

**Higher mode amplification effects in regular ductile RC cantilever walls**

1. The studied parameters affecting the most dynamic shear amplification and seismic force demand are the fundamental lateral period ($T_1$) and the base flexural over-strength ($\gamma_w$) of the wall.

2. While for any $T_1$ dynamic shear amplification significantly reduces with increasing $\gamma_w$, the latter has no significant influence on the shear force demand profile for $T_1 \geq 1.0\text{s}$. Moreover, for $T_1 > 1.0\text{s}$, dynamic shear amplification slightly increases for $\gamma_w \leq 2.0$ and remains almost constant for $\gamma_w \geq 3.0$, as $T_1$ increases.

3. A minimum $\gamma_w$ value of 2.0 can generally preclude the unintended hinge formation at the upper storeys and constrain the plastic mechanism at the wall base, as expected. However, for walls with $2.0 \leq \gamma_w < 3.0$, this observation applies if reinforcement curtailment along the wall height does not result in a flexural strength reduction, between two adjacent storeys, exceeding about 20% to 10% for walls with $T_1$ ranging from 1.0s to 4.0s, respectively.
5.1. CONCLUSION

The proposed capacity design methods produce design strength envelopes in flexure and shear over the height of the wall and are based on the methods proposed by Priestley et al. [2007] and Rutenberg and Nsieri [2006], respectively. New formulations and values are proposed for the parameters defining the envelopes. Among these parameters, there are a dynamic shear amplification factor and the base plasticity height requiring special ductile detailing. The new formulations mainly depend on $T_1$ and the effective force reduction factor, $R_dR_o/\gamma_w$, and hence apply to any design ductility level specified in the NBCC for RC cantilever walls. The complete procedures for determining the proposed envelopes are detailed step by step. The application of the proposed capacity design methods 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 that 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. 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 force amplifications are controlled and mainly resisted by the walls. For shear strength design, a reduced dynamic shear amplification factor accounting for the relative wall participation in the wall-frame system may be used for determining the wall base shear force of the capacity design envelope. The proposed capacity design envelopes represent an upper bound because they do not account for typical 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. A major limitation of the proposed methods is that the proposed values of the parameters defining the envelopes are for seismic regions having a seismic hazard similar to that of the Canadian city of Vancouver located on the West coast. The proposed values may produce unconservative envelopes for eastern Canadian regions such as Montréal because of an underestimation of the higher mode amplification effects, which are generally more significant for the eastern Canadian regions due to a larger high frequency content of their typical earthquake ground motions.
5.2 Future research

This research project has highlighted some issues that need to be investigated and also suggests various future works. Future research projects on the following topics are recommended:

1. The extension of the proposed capacity design methods to eastern Canadian regions;
2. The actual risk of shear failure of ductile and moderately ductile walls designed according to CSA standard A23.3-04 due to high peak shear forces generated by higher mode responses;
3. The applicability of the proposed methods for systems constituted of cantilever walls with largely different cross-sections and lengths;
4. The development of similar capacity design methods for coupled wall systems;
5. The consideration of the foundation and soil type by the proposed methods.
Conclusion et recherche future

5.3 Conclusion

Ce projet de recherche propose pour la norme CSA A23.3 de nouvelles méthodes de dimensionnement à la capacité (DAC) considérant les effets d'amplification des modes supérieurs pour déterminer des enveloppes de dimensionnement en flexion et en cisaillement pour des structures régulières de murs simples ductiles en béton armé à rotule plastique unique (RPU) à la base utilisées comme systèmes de résistance aux forces sismiques (SRFS) de bâtiments à étages. Afin d'atteindre cet objectif, la performance sismique d'un système de murs ductiles en béton armé de 12 étages dimensionné selon le CNBC 2005 et la norme CSA A23.3 2004 est d'abord étudiée. Cette étude visait à évaluer, à l'aide d'un cas réaliste subissant des séismes de dimensionnement et d'autres plus sévères, la validité des méthodes de DAC recommandées par la norme CSA A23.3 2004 pour le dimensionnement en flexion et en cisaillement des murs ductiles en béton armé. Ensuite, une étude paramétrique est menée afin d'identifier l'influence de divers paramètres sur les effets d'amplification des modes supérieurs, et donc sur la demande sismique en force, dans des murs simples ductiles dimensionnés avec le CNBC 2010 et la norme CSA A23.3 2004. L'étude paramétrique est basée sur des analyses dynamiques inélastiques de murs simples encastrés à leur base et modélisés à l'aide de poutres multi-couches, d'une part, et de membranes, d'autre part, prenant en compte la déformation et l'interaction inélastique flexion-cisaillement-force axiale conformément à la théorie du champ de compression modifié et au modèle de champ de contraintes perturbé [Vecchio, 2000]. Cette étude visait à déterminer les paramètres qui influencent le plus la demande sismique en force sur ces murs pour le séisme de dimensionnement et à évaluer de façon plus générale la validité des méthodes de DAC recommandées par la norme CSA A23.3 2004 pour le dimensionnement en flexion et en cisaillement de tels murs. Enfin, une revue de diverses méthodes de DAC proposées dans la littérature et recommandées par des codes de conception pour déterminer des enveloppes de dimensionnement en flexion et en cisaillement pour une conception à RPU est réalisée. Cette revue visait à identifier les limitations des méthodes actuelles à estimer la demande sismique en force sur des murs ductiles dont la réponse sismique en force est contrôlée par les réponses des modes latéraux supérieurs. À partir des résultats de cette revue et de l'étude paramétrique, de nouvelles méthodes de DAC sont proposées et une discussion sur les limitations de ces méthodes et sur leur applicabilité à divers systèmes de murs est présentée.
Du travail réalisé dans ce projet, les principales conclusions sont les suivantes:

**Dimensionnement sismique des murs ductiles en béton armé**

1. Les accélérations spectrales du CNBC 2005/2010 ($S_a(T)$) sous-estiment les réponses élastiques des modes latéraux supérieurs des murs à étages parce que leur amortissement traditionnel de 5% surestime l’amortissement réel (environ 2% en moyenne) et diminue donc les réponses des modes supérieurs.

2. Les méthodes de DAC recommandées par la norme CSA A23.3 2004 pour des murs ductiles peuvent produire des enveloppes de dimensionnement qui sous-estiment la demande en force de cisaillement sur un mur et ne permettent pas de prévenir la formation de rotules plastiques non prévues aux étages supérieurs d’un mur. Ceci est la conséquence d’une déficience des méthodes de DAC qui ne prennent pas en considération les effets d’amplification des modes supérieurs dans le régime inélastique.

3. La relation $0.5l_w + 0.1H$ recommandée par la norme CSA A23.3 2004 pour déterminer la hauteur de plasticité à la base nécessitant une armature spéciale de ductilité est inadéquate puisqu’elle produit des valeurs trop sécuritaires pour des murs élevés dont la surcapacité en flexion à la base est très importante.

4. Une conception à RPU peut être inadéquate et non sécuritaire pour des structures régulières de murs simples ductiles dont le facteur de surcapacité ($\gamma_w$) est inférieur à 2.0 de même que pour des structures ayant des irrégularités en rigidité ou en résistance aux étages supérieurs.

**Effets d’amplification des modes supérieurs dans des structures régulières de murs simples ductiles en béton armé**

1. Les paramètres étudiés les plus influents sur l’amplification dynamique en cisaillement et la demande sismique en force sont la période latérale fondamentale ($T_1$) et la surcapacité en flexion à la base ($\gamma_w$) du mur.

2. Bien que l’amplification dynamique en cisaillement diminue significativement avec l’augmentation de $\gamma_w$ pour n’importe quelle valeur de $T_1$, $\gamma_w$ n’a pas d’influence significative sur le profil de demande en cisaillement pour $T_1 \geq 1.0 s$. De plus, pour $T_1 > 1.0 s$, l’amplification dynamique en cisaillement augmente légèrement pour $\gamma_w \leq 2.0$ et demeure presque constante pour $\gamma_w \geq 3.0$, à mesure que $T_1$ augmente.
5.3. CONCLUSION (FRANÇAIS)

3. Une valeur de $\gamma_w$ d’au minimum 2.0 permet généralement d’éviter la formation de rotules plastiques non prévues aux étages supérieurs et de contraindre le mécanisme plastique à la base du mur, tel que prévu. Toutefois, pour des murs dont $2.0 \leq \gamma_w < 3.0$, cette observation s’applique si la réduction de l’armature le long de la hauteur du mur produit une réduction de la résistance flexionnelle, entre deux étages adjacents, n’excédant pas 20% à 10% respectivement pour des murs dont $T_1$ s’étend de 1.0 s à 4.0 s.

Les méthodes de DAC proposées produisent des enveloppes de dimensionnement en flexion et en cisaillement sur toute la hauteur du mur et sont basées respectivement sur les méthodes proposées par Priestley et al. [2007] et Rutenberg and Nsieri [2006]. De nouvelles formulations et valeurs sont proposées pour les paramètres définissant les enveloppes. Parmi ces paramètres, il y a le facteur d’amplification dynamique en cisaillement et la hauteur de plasticité à la base du mur nécessitant une armature spéciale de ductilité. Les nouvelles formulations dépendent principalement de $T_1$ et du facteur de réduction de force effectif, $R_d R_0 / \gamma_w$, et s’appliquent donc à n’importe quel niveau de ductilité de dimensionnement spécifié dans le CNBC pour les murs simples en béton armé. Les procédures pour déterminer les enveloppes proposées sont détaillées étape par étape. L’application des méthodes de DAC proposées pour le dimensionnement sismique de SRFS concerne essentiellement les murs simples en béton armé qui sont réguliers et uniformes en rigidité et en résistance sur toute la hauteur du bâtiment et qui font partie d’un système agissant comme un mur simple, tel un noyau central dans un gratte-ciel. Cette équivalence de comportement latéral est possible seulement si les murs simples du système ont des sections et des longueurs horizontales similaires et si le système n’est pas irrégulier en torsion. Les méthodes proposées s’appliquent aussi à des murs simples qui font partie de systèmes mur-cadre rigide où les murs contrôlent le comportement latéral du système étant donné que, dans de tels systèmes, l’amplification dynamique des forces est contrôlée et principalement reprise par les murs. Pour le dimensionnement en cisaillement, un facteur d’amplification dynamique en cisaillement réduit prenant en compte la participation relative des murs dans un système mur-cadre rigide peut être utilisé pour déterminer la force de cisaillement à la base de l’enveloppe de DAC. Les enveloppes de DAC proposées représentent une limite supérieure puisqu’elles ne considèrent pas des phénomènes qui peuvent typiquement réduire significativement les forces sismiques reprises par un mur, comme le soulèvement de la fondation, la flexibilité du sol et la contribution en résistance provenant des éléments structurels ne faisant pas partie du SRFS. Une limitation majeure des méthodes proposées est que les valeurs proposées pour les paramètres définissant les enveloppes sont données pour des régions sismiques ayant un risque sismique similaire à celui de la ville de...
Vancouver située sur la côte ouest canadienne. Les valeurs proposées peuvent produire des enveloppes non conservatrices pour des régions de l’est du Canada, telles que Montréal, à cause d’une sous-estimation des effets d’amplification des modes supérieurs, lesquels sont généralement plus importants pour les régions de l’est à cause d’un contenu fréquentiel typiquement plus élevé en hautes fréquences des séismes de ces régions.

5.4 Recherche future

Ce projet de recherche a soulevé certaines questions qui méritent d’être explorées et suggère aussi divers travaux d’investigation futurs. Des projets de recherche futurs sur les thèmes suivants sont recommandés:

1. L’extension des méthodes de DAC proposées aux régions de l’est du Canada;
2. Le risque réel de rupture par cisaillement des murs ductiles et modérément ductiles dimensionnés selon la norme CSA A23.3 2004 attribuable aux forces de cisaillement élevées engendrées par les réponses des modes supérieurs;
3. L’appliquabilité des méthodes proposées pour des systèmes constitués de murs simples dont les sections et les longueurs horizontales sont largement différentes;
4. Le développement de méthodes de DAC similaires pour les systèmes de murs couplés;
5. La prise en compte du type de fondation et de sol par les méthodes proposées.
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