UNIVERSITÉ DE SHERBROOKE
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ASSESSMENT OF THE SEISMIC PERFORMANCE OF A 12-STOREY DUCTILE CONCRETE SHEAR WALL SYSTEM DESIGNED ACCORDING TO THE NBCC 2005 AND THE CSA A23.3 2004 STANDARD

ÉVALUATION DE LA PERFORMANCE SISMIQUE D'UN SYSTÈME DE MUR DE REFEND DUCTILE EN BÉTON DE 12 ÉTAGES DIMENSIONNÉ SELON LE CNBC 2005 ET LA NORME CSA A23.3 2004

Mémoire de maîtrise ès sciences appliquées
Spécialité: génie civil

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Août 2006
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Résumé

Une nouvelle philosophie de conception a été intégrée dans l’édition 2005 du Code national du bâtiment du Canada (CNBC 2005) et dans l’édition 2004 de la Norme canadienne A23.3 pour le dimensionnement des structures en béton (CSA A23.3-04) : le dimensionnement à la performance. Ces codes canadiens incluent de nouvelles dispositions à la performance pour le dimensionnement parasismique des structures de bâtiment. De plus, le CNBC 2005 a été largement révisé depuis l’édition 1995. Bien que les nouvelles dispositions de dimensionnement parasismique du CNBC 2005 et du CSA A23.3-04 n’abordent pas spécifiquement certains problèmes potentiels de dimensionnement des murs de refend ductiles en béton, elles fournissent du moins une approche de dimensionnement plus rationnelle pour de telles structures. Le but de ce projet de recherche consiste à évaluer la performance sismique d’un système de murs de refend ductiles, utilisé comme système de résistance aux charges latérales (SRCL) pour un bâtiment multi-étage en béton armé dimensionné selon le CNBC 2005 et le CSA A23.3-04. Un immeuble de 12 étages situé à Montréal, une ville canadienne, et ayant comme SRCL un système de murs de refend ductiles en béton est dimensionné adéquatement, modélisé en deux dimensions et analysé numériquement à l’aide d’analyses « pushover » et dynamiques inélastiques. Le système de murs se comporte comme un mur en cantilever dans une direction et comme un mur couplé dans la direction orthogonale. Une évaluation de modélisation est réalisée afin d’étudier les effets de divers paramètres de modélisation structurale sur la réponse. Cette évaluation montre que l’influence du raidissement en traction du béton a un effet important sur les prédictions. Les prédictions de l’analyse dynamique indiquent que la demande sismique sur le mur en cantilever est légèrement à considérablement sous-estimée par celle utilisée pour le dimensionnement. En conséquence, l’exigence de la résistance au cisaillement du CSA A23.3-04 n’est pas satisfaite et une rotule plastique additionnelle peut se former au-dessus de la base. La grande différence entre la demande prédite et de dimensionnement est due à une sous-estimation par l’accélération spectrale du CNBC des effets des modes supérieurs dans un mur en cantilever où la réponse sismique est principalement élastique et est dominée par les modes supérieurs. Ce résultat suggère que l’accélération spectrale du CNBC peut être inadéquate pour le dimensionnement parasismique de tels murs.
Abstract

A new design philosophy has been integrated in the 2005 edition of the National Building Code of Canada (NBCC 2005) and the 2004 edition of the Canadian Standard A23.3 for design of concrete structures (CSA A23.3-04): the performance-based design. Consequently, these Canadian codes include new performance-based provisions for the seismic design of building structures. Moreover, the NBCC 2005 has been extensively revised from the 1995 edition. Although the new seismic design provisions of the NBCC 2005 and CSA A23.3-04 do not specifically address some likely design issues regarding ductile concrete shear walls, they provide a more rationale design approach for such structures. The purpose of this research project is to assess the seismic performance of a ductile concrete shear wall system used as seismic force resisting system (SFRS) for a multistorey concrete building designed according to the NBCC 2005 and the CSA A23.3-04. In this purpose, a 12-storey ductile concrete core wall building located in the Canadian city of Montreal is adequately designed, two-dimensionally modelled and numerically analyzed through inelastic pushover and time-history dynamic analyses. The core wall consists of a cantilever wall system in one direction and a coupled wall system in the orthogonal direction. A modelling assessment is performed in order to investigate the effects of different structural modelling parameters on response. This assessment shows that the concrete tension-stiffening effect plays a major role on predictions. The dynamic analysis predictions indicate that the seismic demand at design level on the cantilever wall system is slightly to considerably underestimated by that used for design. As a consequence, the shear strength requirement of the CSA A23.3-04 is not satisfied and an additional plastic hinge formation above the base may be expected. The large difference between the predicted and design demands is essentially due to an underestimation by the NBCC spectral response acceleration of the higher mode effects in a cantilever wall system where the seismic response is mostly elastic and dominated by higher modes. This suggests that the NBCC spectral response acceleration may be inadequate for the seismic design of such systems.
Remerciements

Je souhaiterais transmettre mes plus sincères remerciements et ma plus vive reconnaissance aux personnes suivantes pour leur soutien et leur précieuse collaboration tout au long de la réalisation de ce projet de recherche:

Monsieur Patrick Paultre, mon directeur de recherche, sans qui ce projet n’aurait pas vu le jour. Son expérience, son soutien et ses très judicieux conseils m’ont permis de mener à bon port ce projet. M. Paultre m’a permis de réaliser une étape importante dans mon cheminement professionnel: acquérir le savoir et le savoir-faire pour être un bon ingénieur en structure.

Jean Proulx, Nathalie Roy ainsi que tous les membres du Centre de Recherche en Génie Parasismique (CRGP) de l’Université de Sherbrooke pour leur amitié, leur encouragement et leurs conseils pertinents qu’ils m’ont donnés tout au long de cette recherche.

Karine Veilleux, ma copine, pour son support, sa patience et sa compréhension durant tout ce projet, mais surtout, pour son amour si précieux.

Je souhaite finalement transmettre toute ma gratitude à ma famille et à mes amis qui ont su me comprendre et m’appuyer dans cette aventure.

Je dédie ce travail à mon grand-père, Roger Binette, un homme curieux pour qui l’ingénierie est un domaine où il y aura toujours de quoi à apprendre et s’amuser!
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Chapter 1

Introduction

"Earthquakes systematically bring out the mistakes made in design and construction – even the most minute mistakes; it is this aspect of earthquake engineering that makes it challenging and fascinating, and gives it an educational value far beyond its immediate objective."

NEWMARK AND ROSENBLUETH (1971)

The efficiency of reinforced concrete (RC) shear walls for resisting lateral forces and controlling lateral drift in mid- to high-rise buildings has long been recognized. In high seismically active regions, RC shear walls are the preferred seismic force resisting systems (SFRSs) for all types of buildings. Typical shear walls for multistory buildings are cantilever and coupled shear walls (see Figure 1.1).

It is generally considered both unnecessary and uneconomical to design and construct buildings which will not be damaged under severe earthquakes. In this idea, RC shear walls used as SFRSs are usually designed and detailed in such a way that they will respond in a ductile manner by dissipating inelastic energy through yielding at chosen locations, termed plastic hinges. Such walls are referred to as ductile shear walls or ductile walls.
Recent earthquakes have shown that although buildings designed to current codes performed well from a life safety perspective, the level of damage to structures and the cost of repair were unexpectedly high. To overcome these issues, seismic design codes have decided to move in the direction of limiting damage to acceptable levels as well as protecting life safety. In this perspective, a new more general design philosophy has emerged: the performance-based design.

This new design philosophy has been integrated in the 2005 edition of the National Building Code of Canada (NBCC 2005) and the 2004 edition of the Canadian Standard A23.3 for design of concrete structures (CSA A23.3-04). In consequence, these Canadian codes include new performance-based provisions for the seismic design of building structures. In the CSA A23.3-04, some of these new provisions are specifically for ductile concrete shear wall structures. Moreover, the NBCC 2005 has been extensively revised from the 1995 edition (NBCC 1995). The design-basis earthquake has been increased to a much more severe event than that of the 1995 edition. In addition, dynamic analysis is now the preferred method for seismic design.

Despite all these new enhancements for seismic design in the NBCC 2005 and CSA A23.3-04, currently no published works study the seismic performance of multistorey
CHAPTER 1. INTRODUCTION

ductile concrete shear wall building structures resulting from their application. In this regard, the aim of this research project is to assess from inelastic analyses the seismic performance of a ductile concrete shear wall system used as a SFRS for a multistorey concrete building designed according to the NBCC 2005 and the CSA A23.3-04.

This work includes the following:

Chapter 2: Review of the current state of knowledge of the performance-based design in earthquake engineering as well as of the the new performance-based seismic design provisions of the NBCC 2005 and CSA A23.3-04.

Chapter 3: Review of other new seismic design provisions of these Canadian codes, addressing their application to the design of ductile concrete shear walls.

Chapter 4: Review of the current state-of-the-art methods and modelling technics for 2D inelastic seismic analysis of ductile concrete shear walls.

Chapter 5: Objectives and methodology of this project.

Chapter 6: Design of the studied wall building structure according to the NBCC 2005 and the CSA A23.3-04.

Chapter 7: Modelling assessment and presentation of the structural models used for inelastic seismic analysis.

Chapter 8: Presentation of the inelastic seismic analysis results and important findings.

Chapter 9: Conclusions, recommendations and future research.
Chapter 2

Performance-Based Seismic Engineering

The 2005 edition of the National Building Code of Canada (NBCC 2005) and the 2004 edition of the Canadian Standard A23.3 for design of concrete structures (CSA A23.3-04) include new performance-based provisions for the seismic design of building structures. These provisions are a consequence of the new design philosophy integrated in Canadian Codes as well as in several international building codes for seismic design. This new philosophy is named performance-based design.

The first part of this chapter presents a brief overview of the current state of knowledge of the performance-based design in earthquake engineering. In the second part, the new performance-based seismic design provisions of the NBCC 2005 and CSA A23.3-04 are presented. For the CSA A23.3-04 provisions, only those regarding the flexural ductility of ductile shear walls are addressed in this chapter.
2.1 Performance-Based Seismic Design

2.1.1 Introduction

Severe earthquakes may cause serious damages to buildings even their collapse. However, they are relatively rare events which may or may not ever occur within the life of a building. For such rare earthquakes, it is generally considered both unnecessary and uneconomical to design and construct buildings which will not be damaged. Codes have recognized this in the past and concentrated on life safety as the primary objective of seismic design and not necessarily damage prevention. To reach this main objective, sufficient strength and ductility are provided to buildings using design criteria based on member stresses and forces calculated from a prescribed level of applied lateral shear force. To prevent collapse, building codes prescribe drift limits. However, these limits are established mainly in function of acceptable damages for nonstructural components and not particularly for structural members, though they appear to be reasonable limits for preventing collapse in major earthquakes (NBCC 1995 Commentary).

Recent earthquakes, such as the 1994 M6.7 Northridge and the 1995 M7.2 Kobe earthquakes, have shown that although buildings designed to current codes performed well from a life safety perspective, the level of damage to structures, the economic loss due to business interruption and loss of use, and the cost of repair/reconstruction were unexpectedly high. It was recognized that these costs were too high, even if such events are rare, and that seismic design codes should move in the direction of limiting damage to acceptable levels as well as protecting life safety.

In this new perspective, a more general design philosophy has emerged. This philosophy is the performance-based design and refers to the methodology in which structural design criteria are expressed in terms of achieving a set of performance objectives (Ghobarah, 2001). These objectives are statements of acceptable performance of the structure to be designed and are expressed in terms of expected damage and earthquake hazard levels.

Designing building structures based on performance objective is not something new. A
CHAPTER 2. PERFORMANCE-BASED SEISMIC ENGINEERING

limit state is a form of performance objective. However, the main goal of the performance-based design concept is to develop a general design methodology based on multiple performance objectives and associated earthquake hazard levels. Defining various levels of performance objectives allows control and reduction of costs associated with loss of use and repair of damaged structures after seismic events. The advantage of performance-based design is the possibility of achieving predictable seismic performance with uniform risk.

Performance-based seismic design is part of the more global concept of performance-based earthquake engineering (PBEE). PBEE implies design, evaluation, and construction of engineered facilities whose performance under various seismic load severities responds to the diverse needs and objectives of owners, users and society. As stated by Krawinkler (1999), PBEE is based on the premise that performance can be predicted and evaluated with sufficient confidence for the engineer and client jointly to make intelligent and informed decisions based on building life-cycle considerations rather than on construction costs alone. Although PBEE is an ideal and noble concept, its full implementation is certainly not for tomorrow since several technical, economical, professional and legal challenges need to be addressed and overcome. It will necessitate radical changes in engineering/construction practices and a redirection of research and development (Krawinkler, 1999).

2.1.2 Design Methodology

Figure 2.1 shows the performance-based seismic design methodology, as presented in the SEAOC Vision 2000 Committee’s document (SEAOC 1995). It is noted that the design approach chosen to size and detail the structural system will influence the design procedures. However, the basic design steps and issues considered are similar.

Unlike conventional design methodologies, the first step of this new methodology is the selection of a set of performance objectives for the building based on the client’s expectations, the seismic hazard exposure, economic analysis and acceptable risk. In addition, the selection of performance objectives sets the acceptability criteria for the design and may influence the design approach used. Therefore, performance objectives
Figure 2.1: Performance-based seismic design methodology (SEAOC 1995)
play a major role in this new design methodology. They are the key parameters that control the design process.

This section outlines briefly some of the suggested performance objectives, design approaches and associated acceptability criteria for buildings. Most of the information presented is based upon the SEAOC 1995. This document, the ATC-40 document (ATC 1996) and the FEMA-273 document (FEMA 1997) are credited for laying the foundation for performance-based seismic design concepts.

![Earthquake Performance Level Diagram](image)

*Figure 2.2: Performance objectives for buildings recommended by SEAOC 1995*

**Performance Objectives**

A performance objective is a statement of the expected performance level for the building for a specific earthquake design level. Figure 2.2 shows in a matrix form the minimum design performance objectives recommended by SEAOC 1995 for basic (typical), essential/hazardous (hospitals) and safety critical (nuclear reactors) buildings.

An earthquake design level represents the earthquake ground motion severity for which a particular building performance is desired. It is expressed in terms of a return period in years or a probability of exceedance (ex. 10% in 50 years). A performance level defines the maximum desired extent of post-earthquake damage to a building. Tables 2.1 and 2.2 present descriptions of the performance levels and associated acceptable damage levels.
Table 2.1: Performance levels, damage states and permissible drifts for buildings (SEAOC 1995)

<table>
<thead>
<tr>
<th>Damage Range</th>
<th>Performance Level</th>
<th>Damage State</th>
<th>Permissible Drift</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negligible</td>
<td>Fully Operational</td>
<td>Facility continues in operation without or with negligible structural and non-structural damage</td>
<td>&lt; 0.2%</td>
</tr>
<tr>
<td>Light</td>
<td>Operational</td>
<td>Facility continues in operation with minor damage and minor disruption in non-essential services. Repair is required.</td>
<td>&lt; 0.5%</td>
</tr>
<tr>
<td>Moderate</td>
<td>Life safe</td>
<td>Life-safety is substantially protected, damage is moderate to extensive and structure remains stable.</td>
<td>&lt; 1.5%</td>
</tr>
<tr>
<td>Severe</td>
<td>Near Collapse</td>
<td>Life-safety is at risk, damage is severe, structural collapse is prevented and nonstructural elements may fall.</td>
<td>&lt; 2.5%</td>
</tr>
<tr>
<td>Complete</td>
<td>Collapse</td>
<td>Partial to total structural collapse</td>
<td>&gt; 2.5%</td>
</tr>
</tbody>
</table>

recommended by SEAOC 1995 for buildings and building lateral load resisting systems, respectively, such as concrete shear walls.

Acceptability Criteria

As shown in Figure 2.1, acceptability checks are performed at each design step to verify that the selected performance objectives are being met. The check procedure depends on the performance objectives and design approach used. It may involve both elastic and inelastic analysis methods. Verification is performed using acceptability criteria, which are limit values of any quantifiable structural response parameters such as stress, displacement or deformation. Acceptability criteria can be seen also as performance targets, depending on the design approach taken. Permissible drift limits presented in Table 2.1 are example of acceptance criteria. It is noted that drift limits associated with specific damage states may vary considerably with the structural system and construction material.
<table>
<thead>
<tr>
<th>Structural System</th>
<th>Type</th>
<th>Performance Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral load resisting system</td>
<td>Global structure</td>
<td>Fully Operational: Negligible - generally elastic response; no significant loss of</td>
</tr>
<tr>
<td></td>
<td></td>
<td>strength or stiffness</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Operational: Light - nearly elastic response; original strength and stiffness</td>
</tr>
<tr>
<td></td>
<td></td>
<td>substantially retained. Minor cracking/yielding of structural elements;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>repair implement at convenience.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Life Safe: Moderate - reduced residual strength and stiffness but lateral system</td>
</tr>
<tr>
<td></td>
<td></td>
<td>remains functional.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Near Collapse: Negligible residual strength and stiffness. No story collapse</td>
</tr>
<tr>
<td></td>
<td></td>
<td>mechanisms but large permanent drifts. Secondary structural elements may</td>
</tr>
<tr>
<td></td>
<td></td>
<td>completely fail.</td>
</tr>
<tr>
<td>Concrete shear walls</td>
<td>Primary structure</td>
<td>Negligible: Minor hairline cracking (0.02&quot;) of walls. Coupling beams experience</td>
</tr>
<tr>
<td></td>
<td></td>
<td>cracking &lt; 1/8&quot; width.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Some boundary element distress including limited bar buckling; Some sliding at</td>
</tr>
<tr>
<td></td>
<td></td>
<td>joints; damage around openings; some crushing and flexural cracking; Coupling</td>
</tr>
<tr>
<td></td>
<td></td>
<td>beams - extensive shear and flexural cracks; some crushing, but concrete generally</td>
</tr>
<tr>
<td></td>
<td></td>
<td>remains in place</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Major flexural and shear cracks and voids; sliding at joints; extensive crushing</td>
</tr>
<tr>
<td></td>
<td></td>
<td>and buckling of rebar; failure around openings; severe boundary element damage;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Coupling beams shattered, virtually disintegrated.</td>
</tr>
<tr>
<td>Secondary structure</td>
<td>Negligible</td>
<td>Minor hairline cracking of walls, some evidence of sliding at construction joints.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Coupling beams experience cracks &lt; 1/8&quot; width, minor spalling.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Major flexural and shear cracks; sliding at joints; extensive crushing; failure</td>
</tr>
<tr>
<td></td>
<td></td>
<td>around openings; severe boundary element damage; Coupling beams shattered,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>virtually disintegrated.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Panels shattered virtually disintegrated.</td>
</tr>
</tbody>
</table>
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Generally it is considered that strain and deformation are better indicators of damage than stresses, especially for post-yield damage. For pre-yield damage, however, force- or stress-based criteria may be more appropriate than deformation-based criteria since force variations in the elastic range are usually significantly greater than displacement variations, particularly for short-period structures.

Design Approach

Various design approaches are possible to carry out performance-based seismic design. Among them are force-based and displacement-based approaches. Force-based approaches include the most common seismic design approach adopted in past and current building codes. A displacement-based design approach uses displacement, or deformation, rather than force as the starting point for seismic design. This is based on the idea that control of displacement, or interstorey drift particularly, is the key for controlling the level of damage of the structure and therefore for achieving the desired performance objectives. However, this assumption is an oversimplification since several other parameters, such as failure modes of structural members, earthquake characteristics and accumulation and distribution of structural damage, influence the level of damage (Ghobarah, 2001). In addition, controlling adequately all performance objectives for structural and nonstructural components with a single design parameter such as interstorey drift appears to be unrealistic. Nevertheless, as this approach is well suited to the objectives of performance seismic design, ongoing researches in this field focus on the development of displacement-based design procedures.

2.1.3 Displacement-Based Design Procedures

Among the various proposed displacement-based design procedures (FIB 2003), it appears that the most promising ones are the Direct Deformation-Based Design (DDBD) procedures (Priestley, 1997; Bachmann and Dazio, 1997; Priestley and Kowalsky, 2000). Unlike the others, DDBD procedures are not iterative and do not require a preliminary design. They utilize as a starting point pre-defined target displacements. From these targets, the base shear demand on the building is determined. The design of the structural system
then progresses in a direct manner whereby the end result is the required strength, and hence stiffness, to sustain the target displacements under the specified design earthquake ground motions. This differs significantly from current design code procedures where the maximum displacement demand on the building, due to the prescribed seismic lateral load, is verified at the end of the design process, once the structural system is already designed. As a result, the maximum displacement demand is taken as a design quantity and no attempt is made to alter it if the specified displacement limit is not exceeded.

Although considerable effort is expended to develop displacement-based design procedures for any or specific structural systems and construction materials, currently very few experimental studies (Thomsen and Wallace, 2004) are conducted to verify the reliability of these procedures.

2.1.4 Challenges and Future Trends

Several challenges need to be addressed before procedures for performance-based seismic design can be widely accepted and implemented as seismic provisions in building codes. Ghobarah (2001) presented a summary of these challenges. Some of the most important challenges outlined are the following:

- Development of a general design methodology for multi-performance and earthquake hazard levels, with considerations given on the complete soil-foundation-structure system, nonstructural systems and components and building contents;

- Modelling and analysis of the inelastic behavior and damage of structures for the realistic determination of transient and residual deformations and damage states;

- Validation of the appropriateness of the selected performance levels, the specific parameters used to define their minimum performance states, and the seismic hazard definitions.

One of the main reasons that impede the development of a general performance-based seismic design procedure is the limited ability of the current technology to accurately
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predict the seismic demand and capacity of a structure as well as its level of damage resulting from a specific seismic event. Given these sources of uncertainty inherent to the seismic design process, it is important to recognize that performance-based design will produce structures with predictable performance within certain levels of risk and reliability. For this reason, it is believed (Hamburger, 1999) that performance-based design should be conducted on a probabilistic basis rather than on a deterministic basis.

In this idea, alternative approaches are being developed within the Pacific Earthquake Engineering Research (PEER) Center. Rather than a simple pairing of performance levels and hazard levels, an approach is to compute the probability of reaching or exceeding a certain desired performance and the confidence in characterizing this probability. Performance is expressed in terms of some decision variables, such as repair costs or casualties, that are of interest to decision makers. These kinds of approaches then are promising because they allow a direct understanding of the risk in ways that can be compared with other risks that must be assessed by decision makers (FIB 2003).

2.2 Performance-Based Seismic Design in Canadian Codes

2.2.1 National Building Code 2005

The NBCC 2005 is in an objective-based code format. This is compatible with the international direction of new codes: the performance-based design. In spite of this improvement, the seismic design methodology of the NBCC 2005 does not address yet multiple performance objectives and associated earthquake design levels, as presented in Figure 2.2. Nevertheless, seismic design objectives of the NBCC 2005 have the following intents:

1. to protect the life and safety of buildings occupants and the general public as the building responds to strong ground shaking;

2. to limit damage during low to moderate levels of ground shaking; and
Table 2.3: Overall and interstorey drift limits specified in the NBCC 1995 and 2005

<table>
<thead>
<tr>
<th>Building Type</th>
<th>NBCC 1995</th>
<th>NBCC 2005</th>
</tr>
</thead>
<tbody>
<tr>
<td>Post-disaster building (ex. hospital)</td>
<td>1.0%</td>
<td>1.0%</td>
</tr>
<tr>
<td>School</td>
<td>-</td>
<td>2.0%</td>
</tr>
<tr>
<td>Other</td>
<td>2.0%</td>
<td>2.5%</td>
</tr>
</tbody>
</table>

3. that post-disaster buildings continue to be occupied and functional following strong ground shaking, though minimal damage may be expected in such buildings.

To achieve these objectives, several changes from the NBCC 1995 provisions were made. The main changes related to performance-based seismic design concepts are presented in the following. The provisions follow mainly the recommendations of SEAOC 1995. They are intended to improve the force-based seismic design approach adopted in the NBCC.

The design ground motion (DGM) is now considered to be a very rare event having a probability of exceedance of 2% in 50 years (2500-year return period) at a median confidence level rather than a rare event having a probability of exceedance of 10% in 50 years (475-year return period) as in the NBCC 1995. There is a reasonable degree of confidence that buildings designed with such severe ground shaking will not collapse, though they will likely experience extensive structural and nonstructural damage (NBCC 2005 Commentary). This performance level is termed "extensive damage" and is compatible with the performance level "near collapse" in Table 2.1. The resulting performance objective is similar to that recommended by SEAOC 1995 (see Figure 2.2), except that the associated earthquake design level is based on a 2500-year return period rather than on a 970-year return period. It is expected that well designed and detailed structures will sustain limited structural damage at earthquake shaking levels which are well below the DGM level (NBCC 2005 Commentary).

The damage caused to buildings by earthquake ground motions is a direct consequence of the lateral deformation of the structural system. In the NBCC 2005, the deformation parameter used to assess the potential for structural and nonstructural damage is the interstorey drift. Table 2.3 shows the overall and interstorey drift limits specified in the NBCC 1995 and 2005. In the 2005 edition, a specific drift limit for schools has been added.
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The more stringent drift limit for post-disaster buildings reflects the need for these facilities to remain operational following strong ground shaking. This performance level is termed "immediate occupancy" and is compatible with the performance level "operational" in Table 2.1. Although drift limits specified in the NBCC 2005 appear to be the same or more liberal than those in the NBCC 1995, they are actually more restrictive because they apply to displacements based on a 2500-year return period event rather than on a 475-year return period event. It is noted that specified drift limits are intended to prevent collapse rather than control damage.

Few studies (Heidebrecht and Naumoski, 1999) have investigated the seismic performance, with respect to damage and overall drift levels, of reinforced concrete structures designed in accordance with the Canadian seismic design code provisions. The few studies performed on ductile concrete shear wall systems (Stonehouse et al., 1999) apply to wall systems designed according to the NBCC 1995 and the CSA A23.3-94. As the design level in the 2005 edition of the NBCC has been increased from a 475-year return period event to a 2500-year return period event, new studies are needed for such wall systems designed in accordance with the seismic design provisions of the NBCC 2005 and the CSA A23.3-04.

2.2.2 CSA Standard 2004 for Design of Concrete Structures

A number of provisions in the CSA concrete design standard for the seismic design of reinforced concrete wall buildings have been completely revised for the 2004 edition. Many of these new provisions are displacement-based. For ductile walls, an estimate of the lateral displacement of the building due to the DGM is required in order to assess the flexural ductility of walls and shear strength requirements in plastic hinge regions. The following outlines the new provisions regarding the assessment of the flexural ductility of ductile walls. Shear strength provisions for these structures are discussed in Section 3.2.3 of this document.

Displacement-based provisions for ductility assessment are formulated in terms of inelastic rotations. To ensure that a ductile wall structure has adequate ductility in plastic hinge regions, inelastic rotational capacities \( \theta_{te} \) of this structure must be greater than
CHAPTER 2. PERFORMANCE-BASED SEISMIC ENGINEERING

inelastic rotational demands $\theta_{id}$. Relations presented in this section for calculating $\theta_{ic}$ and $\theta_{id}$ apply to wall structures that are effectively continuous in cross section from the base of structure to top of wall. In addition, they assumed that walls are designed to have a single plastic hinge at their base and that coupled walls are joined together by ductile coupling beams designed to dissipate energy. The rational behind these relations can be found in Adebar et al. (2004) and White and Adebar (2004).

It is noted that the CSA A23.3-04 defines a "fully" coupled wall as a shear wall system with coupling beams, where at least 66% of the base overturning moment resisted by the wall system is carried by the earthquake-induced axial forces in walls resulting from shear in the coupling beams. For lower degrees of coupling, the wall system is defined as a partially coupled wall.

The inelastic rotational demand on a ductile cantilever wall can be determined from:

$$\theta_{id} = \frac{\Delta_f (R_dR_o - \gamma_w)}{h_w - 0.5\ell_w} \geq 0.004$$

(2.1)

where $\Delta_f R_d R_o$ is the design displacement, $\Delta_f \gamma_w$ is the elastic portion of the displacement, $\gamma_w$ is the wall overstrength factor ($\geq 1.3$), $h_w$ is the vertical height of the wall, $\ell_w$ is the horizontal length of the wall in the direction being considered, and 0.004 is a minimum rotational demand. The design displacement is the total lateral displacement at the top of the wall expected for the DGM of the NBCC 2005. It is composed of the total lateral elastic displacement due to the DGM, $\Delta_f$, a ductility-related force modification factor, $R_d$, and an overstrength-related force modification factor, $R_o$. Both previous factors are given in the NBCC 2005 and are explained in Section 3.1.4 of this document. The numerator of Equation (2.1) is the inelastic displacement demand on the wall and the denominator represents the effective height of the wall above the center of the plastic hinge length. This length is assumed to be equal to $1.0\ell_w$. For an entire system of walls acting together, $\ell_w$ should be from the longest wall in the system.

The inelastic rotational capacity of a ductile cantilever wall is given by:

$$\theta_{ic} = \left(\frac{\varepsilon_{cu}\ell_w}{2c} - 0.002\right) \leq 0.025$$

(2.2)
CHAPTER 2. PERFORMANCE-BASED SEISMIC ENGINEERING

where $\varepsilon_{tu}$ must be taken as 0.0035, unless the compression region of the wall is confined as a column, $\ell_w$ is taken as the horizontal length of the individual wall, $c$ is the depth of the neutral axis measured from the compression edge of the wall section, and 0.025 is the upper limit on the inelastic rotation capacity governed by tension steel strain. The tensile strain capacity for bonded reinforcing steel bars embedded in concrete is about 0.05. As the NBCC 2005 limits the maximum global lateral drift to 0.025, the inelastic rotational demand on a concrete wall meeting the NBCC 2005 drift requirements will always be less than 0.025.

The inelastic rotational demand on a ductile coupled wall must be taken as:

$$\theta_{id} = \frac{\Delta f R_d R_o}{h_w} \geq 0.004$$  \hspace{1cm} (2.3)

where $\Delta f R_d R_o$ is the design displacement, $h_w$ is the vertical height of the wall, and 0.004 is the minimum rotational demand. The inelastic rotational capacity $\theta_{ic}$ of a wall segment must be calculated using Equation (2.2), except that the wall length $\ell_w$ to be used is the individual wall segment length for a partially coupled wall and the length of the entire system for a fully coupled wall.

The inelastic rotational demand on coupling beams must be taken as:

$$\theta_{id} = \left( \frac{\Delta f R_d R_o}{h_w} \right) \frac{\ell_{cg}}{\ell_u}$$  \hspace{1cm} (2.4)

where $\ell_{cg}$ is the horizontal distance between the centroids of the walls on either side of the coupling beams, and $\ell_u$ is the clear span of these beams. The ratio between brackets in Equation (2.4) is the inelastic rotational demand on the coupled wall, which is given by Equation (2.3). The inelastic rotational capacity of coupling beams $\theta_{ic}$ must be taken as:

(a) 0.04 for coupling beams designed with diagonal reinforcement (Fig. 2.3a); and
(b) 0.02 for coupling beams designed with conventional reinforcement (Fig. 2.3b).
Figure 2.3: Typical diagonally and conventionally reinforced coupling beam
Chapter 3

Canadian Seismic Provisions for Ductile RC Walls

Section 2.2 of this document outlines the new performance-based seismic design provisions of the NBCC 2005 and those of the CSA A23.3-04 for the assessment of the flexural ductility of ductile concrete shear walls. In this chapter, other new seismic design provisions of these Canadian Codes are presented. Application of these provisions to the design of buildings having ductile concrete shear walls as seismic force resisting systems (SFRSs) is addressed in this chapter.
3.1 National Building Code 2005

3.1.1 General Requirements

The NBCC 2005 now requires that building structures be designed with clearly defined load path(s) to transfer inertial forces generated in an earthquake to the supporting ground. The elements in the load path(s) must be clearly identified as being the seismic force resisting system (SFRS). They must be designed to resist 100% of the earthquake loads. All elements of the structural system not considered as part of the SFRS must displace with the building to its expected earthquake displacement and resist the loads induced by this displacement while preserving their vertical load carrying capacity.

3.1.2 Design Loading Cases

In the NBCC 2005, the design loading cases for ultimate limit states are specified in terms of load combinations, which are composed of principal and companion loads. One of the combinations that must be considered is the principal loads acting alone. The factored loads to be used for seismic design must be determined in accordance with the load combinations presented in Table 3.1. From this table, it is noted that the earthquake load \((E)\) is factored by 1.0. This is because the NBCC considers the earthquake load as a rare event. The rationale behind these load combinations and factors can be found in Bartlett et al. (2003).

3.1.3 Methods of Analysis

In the NBCC 2005, two methods of analysis can be carried out to determine the earthquake design actions in building structures. These methods are the Equivalent Static Force Procedure (ESFP), which is the common static method used in most building design codes, and the Dynamic Analysis Procedure (DAP). In contrast with the previous editions
Table 3.1: Load combinations specified in the NBCC 2005 for seismic design

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>Principal Loads</th>
<th>Companion Loads</th>
<th>Use and Occupancy</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$1.0D + 1.0E$</td>
<td>-</td>
<td>All</td>
</tr>
<tr>
<td>2</td>
<td>$1.0D + 1.0E$</td>
<td>$+ 1.0L + 0.25S$</td>
<td>For storage occupancies, equipment areas and service rooms</td>
</tr>
<tr>
<td>3</td>
<td>$1.0D + 1.0E$</td>
<td>$+ 0.5L + 0.25S$</td>
<td>For other occupancies</td>
</tr>
</tbody>
</table>

$D$: Dead load; $E$: Earthquake load; $L$: Live load; $S$: Snow Load

of the NBCC, the default method of analysis in the NBCC 2005 is the DAP. The general rationale for this change is that structures respond to earthquakes dynamically rather than statically. It is well known that dynamic analyses provide better estimations of earthquake actions in building structures, especially in multistory buildings. The ESFP may be used only if any of several specified criteria are met. These criteria are relative to the seismic hazard of the design spectral acceleration and the height, the fundamental lateral period of vibration and the structural regularity of the building. Independently of the method of analysis used, a minimum lateral (horizontal) earthquake force at the base of the building must be calculated before carrying out any analysis.

### 3.1.4 Minimum Lateral Earthquake Force

The NBCC requires that a building be designed to resist without collapse a minimum lateral earthquake force at its base. In the NBCC 1995, this force, $V$, is given by:

$$V = (V_e/R)U$$  \hspace{1cm} (3.1)$$

where $V_e$ is the equivalent lateral force at the base of the structure representing elastic force, $R$ is the force modification factor, which reflects the capability of a structure to dissipate energy through inelastic behavior, and $U$ is a calibration factor ($=0.6$) to calibrate the seismic design force values to historical levels. The force $V_e$ is determined from the product of the zonal velocity ratio ($v$), determined for 10% probability of exceedance in
50 years, the seismic response factor \( S \), the foundation factor \( F \), the importance factor \( I \) and the seismic weight \( W \) of the building. Detailed explanations on each previous parameter can be found in the NBCC 1995.

In the NBCC 2005, the minimum lateral earthquake force, \( V \), also referred to as the design base shear for the ESFP, is given by the following equation:

\[
V = \frac{S(T_a)M_oI_EW}{R_dR_o} \geq \frac{S(2.0)M_oI_EW}{R_dR_o} \tag{3.2}
\]

where \( S(T_a) \) is the design spectral response acceleration, expressed as a ratio of gravitational acceleration, for the fundamental lateral period of vibration, \( T_a \), of the building; \( M_o \) is a 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, which is composed of the dead load of the building plus 25\% of the design snow load plus 60\% of the storage load and the full contents of any tanks; \( R_d \) is a ductility-related force modification factor that reflects the capability of the structure to dissipate energy through inelastic behavior; and \( R_o \) is an overstrength-related force modification factor that accounts for the dependable portion of reserve strength in the structure designed according to NBCC provisions. For a building having a SFRS with a \( R_d \geq 1.5 \), \( V \) in Equation (3.2) need not be taken greater than \((2/3)S(0.2)I_EW/(R_dR_o)\).

The product \( S(T_a)M_oI_EW \) in Equation (3.2) represents the maximum lateral seismic force, \( V_o \), at the base of an elastic system with the fundamental period \( T_o \). As shown in Figure 3.1, this product is divided by the force reduction factor \( R_dR_o \) in order to reduce the earthquake design forces of buildings for which seismic force resisting systems are designed to dissipate inelastic energy. The NBCC recognizes explicitly that seismic forces are reduced when structural response goes into the inelastic range. Through inelastic energy dissipation, the NBCC expects that a high degree of life-safety protection be achieved (NBCC 2005 Commentary).

Brief explanations of the parameters of Eq. (3.2) are presented in the following, emphasizing on their application to the design of ductile concrete shear walls. Comparisons between these parameters and those of Eq. (3.1) are established when it is pertinent. A more detailed comparison between Eq. (3.2) and (3.1) can be found in Heidebrecht (2003).
Fundamental Lateral Period of Vibration

The determination of $T_a$ for the direction of loading under consideration depends on the type and the construction material of the SFRS of the building in this direction. For a concrete shear wall building, the NBCC 2005 requires that $T_a$ (in seconds) be determined with the following empirical equation:

$$T_a = 0.05 \left( h_n \right)^{3/4} \quad (3.3)$$

where $h_n$ is the height above the base of the building in meters. The NBCC permits that established methods of mechanics, such as a linear modal analysis, be used to determine $T_a$. However, the NBCC now requires that these methods use a structural model that complies with specific structural modelling requirements. Structural modelling must be representative of the magnitude and spatial distribution of the mass of the building and of the stiffness of all SFRS members. In addition, it must account for various effects, such as cracked sections in reinforced concrete (RC) members, that influence the building lateral stiffness. Only for the purpose of determining $T_a$, structural elements not part of the SFRS must be accounted for in the structural modelling if the added stiffness decreases the computed $T_a$ by more than 15%. As computed periods tend to be longer than those measured in actual buildings, the NBCC specifies that the computed $T_a$ of a RC shear wall structure must not be greater than 2.0 times that calculated with Equation (3.3).
Design Spectral Response Acceleration

The design spectral response acceleration, $S(T)$, is determined as follows:

$$S(T) = F_a S_a(T) \text{ or } F_v S_a(T), \text{ depending on } T$$  \hspace{1cm} (3.4)

where $S_a(T)$ is the 5% damped spectral response acceleration, determined for a probability of exceedance of 2% in 50 years at a median confidence level, of an elastic single-degree-of-freedom (SDOF) system with period $T$, and $F_a$ and $F_v$ are the acceleration-based and velocity-based site coefficients, respectively. Both site coefficients represent the amplification of seismic motions due to ground conditions. Softer is the ground condition higher is the amplification. The NBCC 1995 includes only one site (foundation) factor ($F$). The value of this factor depends on ground conditions only. The values of the new period-dependent site coefficients specified in the NBCC 2005 are a function of ground conditions and the intensity of ground motion. They vary between 0.7 and 2.1 for $F_a$ and 0.5 and 2.1 for $F_v$. Both site coefficients equal to 1.0 for a very dense soil or soft rock condition. This is the reference ground condition (RGC) on which are based the $S_a(T)$ values specified by the NBCC 2005. In the NBCC 1995, the RGC is rock or stiff soil. This RGC is now broken into three different ground conditions in the NBCC 2005: hard rock, rock and very dense soil or soft rock (new RGC).

It is noted that $S_a(T)$ is not an acceleration response spectrum, but essentially a uniform hazard spectrum (UHS). A UHS is a plot of spectral acceleration ordinates at different periods calculated at the same probability of exceedance and for a given damping. The spectral ordinates are determined at specific geographical locations of interest, such as towns or cities. The spectral values at different periods may arise from earthquakes having different magnitudes and distances from a specific site, but the same annual probability of exceedance. A UHS can be considered as the envelope of maximum spectral acceleration values produced by different earthquakes. This differs significantly from the classical response spectrum of a single earthquake.

Adams and Atkinson (2003) developed seismic hazard maps for use with the NBCC 2005. These maps provide UHS ($S_a(T)$) values for specific periods, which are 0.2s, 0.5s, 1s and 2s, and for a selected number of Canadian cities across the country. From these
UHS values, spectra closely matching the shape of the uniform hazard spectra can be constructed, adjusted for ground conditions and used for seismic design. Figure 3.2 shows the NBCC 2005 acceleration spectrum, constructed from the four specified UHS values, for the city of Montréal. For this city located in the eastern part of Canada, \( S_a(T) \) values are associated to a peak ground acceleration (PGA) value of 0.43g. In the NBCC 1995, the design PGA value, which corresponds to a probability of exceedance of 10% in 50 years, is 0.18g for Montréal. This comparison shows that the PGA value of the NBCC 2005 design ground motion for this city is almost 2.4 times that of the previous code edition.

![Figure 3.2: NBCC 2005 acceleration spectrum for the city of Montréal](image)

**Higher Mode Effects**

The product \( S(T_o)I_oW \) in Equation (3.2) represents the maximum lateral seismic force in an elastic SDOF system with the fundamental period \( T_o \), as \( S(T) \) is essentially the response spectrum of an elastic SDOF system. Using only this force and the force reduction factor \( R_dR_o \) to calculate the design base shear \( V \) means that the elastic dynamic response of a building, which is a multi-degree-of-freedom (MDOF) system, can be represented only by its first flexural mode response at \( T_o \). This assumption yields relatively good approximations for short-period structures. However, for long-period structures, such as multistorey buildings, higher mode effects tend to increase base shear from that calculated for an elastic SDOF system. For an elastic structure, these effects depend on the shape of the design spectrum and the dynamic characteristics of the structure, which are the natural periods and the effective modal weights (Humar and Rahgozar, 2000). To account explicitly for the higher mode effects on base shear for elastic structures, the NBCC 2005 introduces the new \( M_o \) factor in Equation (3.2). The derivation of this factor can be found
Table 3.2: Higher mode factor $M_v$ specified in the NBCC 2005 for RC shear walls.

<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$^{(1)}$</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>$\geq 8.0$</td>
<td>Coupled wall$^{(1)}$</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>

(1) Coupled wall is a shear wall system with coupling beams where 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.

in Humar and Mahgoub (2003). In the NBCC 1995, higher mode effects are included in the seismic response factor $S$, which is a function of the fundamental lateral period of vibration of the building and a zonal acceleration ratio. This factor is equivalent to the product $M_v S_a(T_a)$ in the NBCC 2005.

The $M_v$ values specified in the NBCC 2005 are a function of the type of SFRS, the fundamental period $T_a$ of the building and the shape of the spectral response acceleration $S_a(T)$. Table 3.2 presents the $M_v$ values specified in the NBCC 2005 for RC shear walls. A ratio $S_a(0.2)/S_a(2.0) < 8.0$ is typical for the western regions of Canada while a ratio $S_a(0.2)/S_a(2.0) \geq 8.0$ is typical for the eastern regions of the country. It is noted that higher mode effects increase with $T_a$. This is due to a greater contribution of the higher modes to base shear for long-period structures than for short-period structures.

The $M_v$ factor is determined from an elastic modal response spectrum analysis. Dynamic magnification on base shear due to inelastic effects of higher modes is not taken into account in $M_v$. This factor then differs from the dynamic shear magnification factor specified in the New Zealand’s concrete design standard (NZS 1995) and recommended in the CAC explanatory notes (CAC 1995) on the 1994 Canadian concrete design standard (CSA A23.3-94) to account for inelastic effects of higher modes for shear design of ductile concrete shear walls. The use of this factor is discussed further in Section 3.2.3 of this chapter. If a $M_v$ factor is to be determined for an inelastic structure, Humar and Rahgozar (2000) stated that this factor must be selected so that the highest ductility demand in the MDOF system is equal to or less than the target ductility for which the base shear for the associated SDOF system is calculated. In fact, their study showed from
Table 3.3: $R_d$ and $R_o$ values specified in the NBCC 2005 for ductile concrete shear walls

<table>
<thead>
<tr>
<th>Type of SFRS</th>
<th>NBCC</th>
<th>Values used to derive $R_o$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$R_d$</td>
<td>$R_o$</td>
</tr>
<tr>
<td>Ductile shear wall</td>
<td>3.5</td>
<td>1.6</td>
</tr>
<tr>
<td>Ductile partially coupled wall</td>
<td>3.5</td>
<td>1.7</td>
</tr>
<tr>
<td>Ductile &quot;fully&quot; coupled wall</td>
<td>4.0</td>
<td>1.7</td>
</tr>
</tbody>
</table>

Inelastic dynamic analyses that the ductility demands for MDOF systems may differ significantly from those of the associated SDOF systems, for the same base shear strengths. Therefore, the design base shear must be adjusted to ensure that the maximum ductility demand in the MDOF system is kept within the target ductility. Currently no $M_o$ values are proposed in the available literature for inelastic structures.

**Force Modification Factors**

The ductility-related force modification factor, $R_d$, in Equation (3.2) corresponds essentially to the $R$ factor in Equation (3.1). The $R_d$ values specified in the NBCC 2005 range from 1.0 for brittle systems, such as unreinforced masonry, to 5.0 for the most ductile systems, such as ductile steel plate walls. For concrete SFRSs designed and detailed according to the CSA A23.3-04, the $R_d$ values range from 1.0 to 4.0. This range is believed to be realistic for MDOF concrete structures (Paulay and Priestley, 1992). Table 3.3 presents the $R_d$ values specified in the NBCC 2005 for ductile concrete shear walls. These values are the same as those given in the NBCC 1995.

The NBCC 2005 includes a new overstrength-related force modification factor, $R_o$, to explicitly account for the dependable portion of reserve strength arising from the application of the design and detailing provisions prescribed in the associated CSA design standards. In lieu of increasing the factored resistance to account for overstrength, the design force level is reduced by including the $R_o$ factor in the denominator of Equation (3.2). Mitchell et al. (2003) stated that this approach is more in line with usual design procedures where the factored resistance is compared with the factored load effects as obtained from linear analysis.
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To account for the various components contributing to $R_o$, the following formulation was chosen:

$$R_o = R_{size} R_\phi R_{yield} R_{sh} R_{mech}$$  \hspace{1cm} (3.5)

where $R_{size}$ is the overstrength arising from restricted choices for sizes of members and elements and rounding of sizes and dimensions; $R_\phi$ is a factor accounting for the difference between nominal and factored resistances, equal to $1/\phi$, where $\phi$ is the material resistance factor as defined in the CSA standards; $R_{yield}$ is the ratio of "actual" yield strength to minimum specified yield strength; $R_{sh}$ is the overstrength due to the development of strain hardening in the material at the anticipated level of deformation of the structure; and $R_{mech}$ is a factor to account for the additional resistance that can be developed before a collapse mechanism forms in the structure. The $R_o$ values specified in the NBCC 2005 for ductile concrete shear walls are presented in Table 3.3 as well as the values of the factors used to derive $R_o$. The rationale behind these values is explained in Mitchell et al. (2003).

3.1.5 Dynamic Analysis Procedure

As indicated in Section 3.1.3 of this chapter, the default method of analysis in the NBCC 2005 for determining earthquake design actions in building structures is the Dynamic Analysis Procedure (DAP). The code requires that the DAP be in accordance with one of the following methods:

a) A linear (elastic) dynamic analysis by either a modal response spectrum method or a numerical integration linear time-history method using a structural model that complies with the structural modelling requirements specified in the NBCC;

b) A nonlinear (inelastic) time-history dynamic analysis method, in which case a special study must be performed. This method is addressed in Chapter 4 of this document.

The preferred linear method is the modal response spectrum method as it is simple and now a straightforward procedure. Actually conducting a mode superposition analysis of a linear elastic system is facilitated by the fact that seismic hazard now is specified in terms
of spectral acceleration. The NBCC 2005 requires that the spectral acceleration values used in this method of analysis be the design spectral acceleration values of $S(T)$. The design response spectrum is constructed from the UHS values specified for the site and the site coefficients $F_a$ and $F_v$. The use of UHS values as the design response spectrum for a MDOF system is conservative and appropriate for seismic design (Humar and Mahgoub, 2003). There are no requirements in the NBCC 2005 or recommendations in the NBCC 2005 Commentary on the method that should be used to combine the maximum modal response values of the considered modes. For this purpose, the NBCC 1995 Commentary recommends the square-root-of-the-sum-of-the-squares (SRSS) method when modal periods are well separated or the complete quadratic combination (CQC) method when some modal periods are closely spaced or identical. Generally only a small number of modes (3 to 5) are required to provide a relatively good approximation of the total response. The NBCC 2005 Commentary provides a simple rule for the determination of the number of modes required: at least 90 percent of the participating mass of the structure must be included in the calculation of response for each principal direction. Therefore, the number of modes to be considered must satisfy this requirement.

If the numerical integration linear time-history method of dynamic analysis is used, the NBCC 2005 requires that the earthquake ground motion histories be compatible with a response spectrum constructed from the design spectral acceleration values of $S(T)$. This requirement also applies to the nonlinear time-history dynamic analysis method. The NBCC 2005 Commentary states that a ground-motion time history is deemed to be "spectrum-compatible" if its response spectrum equals or exceeds the target spectrum throughout the period range of interest, i.e. the periods of the modes contributing to the response of the structure. Spectrum-compatible time histories may be obtained by scaling and/or modifying actual recorded earthquake accelerograms obtained from site-specific earthquakes or by creating simulated ground-motion time histories having amplitudes and frequency content that are consistent with seismological observations for earthquakes within the specific regions. Simulated time histories are often required because of the limited number of actual records available for such purposes. In this idea, Atkinson (1999) produced simulated ground-motion time histories, which are compatible with the uniform hazard spectra provided by the NBCC 2005 seismic hazard maps, for several Canadian cities. As a UHS does not represent a single earthquake, more than one simulated ground-motion time history are required to match a UHS. Atkinson and Beresnev showed that a
UHS is adequately matched with just two types of earthquake: a lower magnitude, smaller
distance earthquake to match the short period part of the spectrum and a larger mag-
nitude, greater distance earthquake to match the long period part. A sufficient number
of ground-motion time histories must be used to enable uncertainties in ground motion
parameters to be reflected in the dispersion of the resulting response parameters.Actually
response history analyses, particularly inelastic analyses, are highly dependent on the
characteristics of the individual ground motion records and minor changes in these records
can lead to significant differences with regard to the predicted response of the structure.
The NBCC 2005 Commentary suggests the recommendations of the FEMA-450 docu-
ment (FEMA 2003) with regard to the number of records to be considered when response
history analysis is used for design. The provisions of this document require a suite of
not fewer than three appropriate ground-motion time histories. Also they require that
the maximum values of the predicted response parameters be used as the design values
when a suite contains fewer than seven records. When seven or more records are used,
then mean values of the response parameters may be used. This can lead to a substantial
reduction in design forces and displacements and typically will justify the use of larger
suites of records.

The effects due to the torsional eccentricity of a (asymmetric) structure and the acci-
dental torsion must be accounted for in the determination of the seismic design actions.
The accidental torsion refers to torsions that may arise from the undetermined asymme-
try of a building and the rotational motion of the ground about the vertical axis (Humar
et al., 2003). Generally the effects due to the torsional eccentricity can be well accounted
for using a three-dimensional dynamic analysis. However, structural modelling for such
analysis does not ordinarily take into account the effects of the accidental torsion. The
NBCC 2005 provides two methods to account for these effects, which must be considered
to act concurrently with the effects of the lateral motion including actual eccentricities.
One method uses a static approach while the other method uses a dynamic approach. In
both methods, the accidental torsion is simulated by introducing a hypothetical eccentric-
ity in the structural model. The dynamic method is permitted only for structures that
are not torsionally sensitive, that is, for $B<1.7$, where $B$ is the new torsional sensitivity
parameter as defined in the NBCC 2005.
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The elastic base shear, $V_e$, obtained from the linear dynamic analysis must be multiplied by the factor $I_E/(R_d R_o)$ to take into account the inelastic response and the importance of the structure. The resulting base shear, $V_d$, referred to as the design base shear for the DAP, is on a comparable basis to the design base shear $V$ determined with Equation (3.2). It is noted that the higher mode effects on base shear are intrinsic to $V_d$. As structural models tend to be more flexible than actual structures, $V_d$ may be lower than $V$. To ensure a minimum level of base shear for design, the NBCC 2005 requires that $V_d$ be taken as 0.8$V$ if it is less than 80% of $V$. For irregular structures requiring the DAP as method of analysis (see Section 3.1.3 of this chapter), the Code requires that $V_d$ be taken as $V$ if it is less than $V$. The values of the design actions, such element forces, are obtained by multiplying the elastic actions obtained from the linear dynamic analysis by the ratio $V_d/V_e$. The resulting deflections and interstorey drifts must be multiplied by the factor $R_d R_o / I_E$ to obtain the design values, which are more realistic values of the anticipated maximum deflections and drifts.

3.2 CSA Concrete Standard A23.3 2004

3.2.1 Effective Properties for Seismic Analysis

The NBCC 2005 now requires that earthquake design actions, such as element forces and deflections, determined from a linear elastic analysis, be obtained from structural models that comply with specific structural modelling requirements. One of these requirements is that elastic structural models must account for the effect of cracked sections in RC members. This requirement also applies if the fundamental lateral period of vibration of the building, $T_o$, is determined using a linear modal analysis. For these purposes, the CSA A23.3-04 now specifies effective (cracked) section properties for elastic models of RC structures. Those for wall structures are presented in Table 3.4, as fractions of gross (uncracked) section properties. In the previous edition of this standard, no effective section properties are specified for the seismic design of RC structures. These section properties are provided in the CAC explanatory notes (CAC 1995) on this standard. Their expression
and value differ from those presented in Table 3.4.

The effective wall section properties presented in Table 3.4 are a function of the reduction factor $\alpha_w$, which is given by:

$$\alpha_w = 0.6 + \frac{P_g}{f'_c A_g} \leq 1.0$$ (3.6)

where $P_g$ is the axial compression force at the base of the wall due to gravity loads, $f'_c$ is the specified compressive strength of concrete, and $A_g$ is the gross area of the wall section at the base. For multiple wall segments, a single reduction factor based on an average value of the axial compression stress, $P_g/A_g$, may be used to estimate the overall effective stiffness of the structure.

Equation (3.6), originally proposed by Paulay (1986), shows that the reduction factor for a RC shear wall is a function of the level of axial compression force at its base. In fact, Ibrahim and Adebar (2004) found that the level of axial compression force at the base of a RC wall has the greatest influence on the elastic flexural rigidity of the entire wall, compared to the amount of vertical reinforcement, concrete compressive strength, yield strength of vertical reinforcement and shape of the wall section. They also found that Equation (3.6) is appropriate to estimate the effective stiffness of a RC wall structure that is essentially uncracked before being yielded. It is, however, of interest to note that the relationship between effective stiffness and concrete compressive strength in Equation (3.6) is inadequate for such walls. In fact, Equation (3.6) predicts for a given axial compression load that effective stiffness decreases with increasing concrete compressive strength. This leads to unrealistic low estimates of the effective stiffness for walls made of high-strength
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Concrete.

It is noted that $P_e$ in Equation (3.6) is due to gravity loads only. The effect of the earthquake-induced axial forces in a coupled wall resulting from the coupling beam shear forces is ignored for simplicity. Although these forces reduce the compression in one wall and increase the compression in the adjoining wall, the average axial compression forces on both walls do not change.

More details on the rationale behind the effective section properties presented in Table 3.4 can be found in Adebar et al. (2004) and Ibrahim and Adebar (2004).

3.2.2 Seismic Design of Ductile Walls

As linear elastic analyses are generally used to predict earthquake design actions in concrete building structures, the seismic design provisions of the CSA A23.3-04 are based on capacity design principles (Park and Paulay, 1975). In a capacity design, the designer chooses the inelastic mechanism of the SFRS when the building structure undergoes inelastic deformation. Appropriate strengths are provided to the critical regions of the SFRS members chosen to dissipate inelastic energy to ensure that the selected mechanism will form. These critical regions, termed plastic hinges, are specially designed and detailed for inelastic flexural action, and all potential brittle failure modes, such as shear failure, are inhibited by suitable strength differential and detailing. To prevent their failure, the strength of all other structural members must be greater than that corresponding to development of maximum feasible strength in the potential plastic hinge regions.

The seismic design provisions in the CSA A23.3-04 for ductile walls are developed for wall structures that are substantially uniform and regular in strength and stiffness over the full height of the building. For such ductile wall structures, it is assumed that plastic hinges will develop as shown in Figure 3.3, that is, at wall base for cantilever walls and at coupling beam ends and wall base for coupled walls. These locations are obtained by assuming that plastic hinges will develop where the bending moments in the wall structure members are maximum under a pushover-type loading. This loading deforms the structure similarly to its first flexural mode of vibration. Therefore, plastic hinge locations other
than those shown in Figure 3.3 are likely if the seismic response of the structure is dominated by higher modes (Seneviratna and Krawinkler, 1994).

Specific design requirements must be applied and a special detailing must be provided at plastic hinge locations to ensure the development of the inelastic mechanism. For the base plastic hinge, the CSA A23.3-04 requires that this detailing be provided above the design critical section (see Fig. 3.4 in this chapter) for a height equal to at least 1.5 times the plastic hinge height, which is assumed to be equal to the wall length, \( l_w \), or the length of the longest wall in the direction under consideration. The resulting detailing height is noted \( l_{pd} \) and represents the base plastic hinge region. Detailing for plastic hinging must extend below the critical section to the footing or the distance \( l_{pd} \), depending on the level of strength and stiffness below the critical section.

For a capacity design, appropriate strength must be provided to a wall to prevent premature flexural yielding above the base plastic hinge region. For this purpose, the CSA A23.3-04 provides new capacity design provisions to determine a capacity moment demand corresponding to the development of the factored moment resistance of the wall section over the assumed hinging region. The factored moment resistance \( (M_e) \) takes into account the reduced strengths, as specified in the CSA A23.3-04, of the concrete.
and reinforcing steel bars of a wall section. The capacity moment demand in a wall is determined by increasing the NBCC design (factored) overturning moments obtained from linear analysis by the ratio of \( M_r \) to the factored moment, \( M_f \), both calculated at the top of the assumed hinging region. The resulting demand is shown in Figure 3.4. An alternative approach, suggested in the CAC 1995, is to assume a probable moment demand varying linearly from the top of the plastic hinge region to the top of the wall, as illustrated in Figure 3.4. The CSA A23.3-04 provides additional requirements for ductile coupled walls to ensure that the wall capacities will be sufficient to produce the desired inelastic mechanism, i.e. yielding of coupling beams prior to walls.

![Diagram of moment demands for ductile concrete shear walls](image)

**Figure 3.4:** Capacity design moment demands for ductile concrete shear walls

If the wall structure contains vertical stiffness or geometric irregularities, as defined in the NBCC 2005, over its height, the detailing for plastic hinging must be applied over each irregularity and must continue for the distance \( \ell_{pd} \) above and below each irregularity.

For wall structures that do not meet the structural conditions aforementioned in this
section, the CSA A23.3-04 requires that these walls be detailed for plastic hinges to occur at all expected locations. To determine these locations, inelastic seismic analysis may be needed.

For ductile coupled walls, the CSA A23.3-04 requires that ductile coupling beams be provided and have a depth no greater than two times the clear span of the beam. Coupling beams reinforced with diagonal reinforcement must be provided. However, if the factored shear stress in a beam is less than a certain limit, either diagonal or conventional reinforcement must be provided. A diagonal reinforcement has the advantage of providing a greater ductility and resisting both in-plane flexural and shear loads in the beam. The design and detailing requirements prescribed by the CSA A23.3-04 for diagonally reinforced coupling beams are essentially the same as those prescribed by the CSA A23.3-94. They are based on the recommendations in Paulay and Priestley (1992). Most of these recommendations come from the work of Paulay and Binney (1974). The proportioning of the diagonal reinforcement is based on the assumption that diagonal reinforcement resists the entire in-plane loads after yielding and load reversal, and therefore that each diagonal member carries the same magnitude of axial force. The contribution of slab reinforcement is assumed negligible for beam design. The beam reinforcement yielding strength should match the shear demand as closely as possible in order to avoid any subsequent difficulties in the design of wall segments and foundations.

3.2.3 Shear Design of Ductile Walls

Design Shear Resistance

The CSA A23.3-04 requires that the factored shear resistance of a non-prestressed wall reinforced in shear with horizontally-disposed transverse reinforcement be taken as:

\[
V_r = V_c + V_s = \phi_c \lambda \beta \sqrt{f'_{c}} b_w d_v + \frac{\phi_s A_v f_y}{s} d_v \cot \theta \leq 0.25 \phi_c f'_{c} b_w d_v \tag{3.7}
\]

where \(V_c\) and \(V_s\) are the factored shear resistance provided by the concrete and the transverse reinforcement, respectively; \(\phi_c = 0.65\) and \(\phi_s = 0.85\) are the resistance factors for
concrete and reinforcing steel bars, respectively; $\lambda$ is a factor to account for low-density concrete ($\lambda=1.0$ for normal density concrete); $\beta$ is a factor accounting for shear resistance of cracked concrete; $f'_c$ is the specified compressive strength of concrete; $b_w$ is the minimum wall thickness within depth $d_v$; $d_v$ is the effective shear depth of the wall and needs not be taken less than $0.8\ell_w$; $A_v$ is the area of transverse reinforcement within a distance $s$; $s$ is the spacing of transverse reinforcement measured along the longitudinal axis of the wall; $f_y$ is the specified yield strength of the reinforcing steel bars; and $\theta$ is the angle of inclination of diagonal compressive stresses to the longitudinal axis of the wall.

Under a major earthquake loading, plastic hinge regions will be damaged during the reverse cyclic inelastic rotations that they will experience. To account for this damage, the CSA A23.3-04 provides specific shear strength requirements for the plastic hinge regions of ductile walls. These requirements are based on the inelastic rotational demand $\theta_{id}$ of a wall, as it is a good indicator of damage in plastic hinge regions of RC shear walls (CAC 2005). A requirement reduces the maximum factored shear resistance, $V_{r,\text{max}}$, of Equation (3.7) from $0.25\phi_c f'_c b_w d_v$ to $0.10\phi_c f'_c b_w d_v$ unless $\theta_{id}$ is less than 0.015. When $\theta_{id} \leq 0.005$, $V_{r,\text{max}}$ is limited to $0.15\phi_c f'_c b_w d_v$. For a $\theta_{id}$ value between these limits, linear interpolation may be used. Other requirements limit the values of $\beta$ and $\theta$ in Equation (3.7).

**Design Shear Demand**

The CSA A23.3-04 requires that the factored shear resistance, $V_r$, of a ductile wall system be not less than the lesser of:

a) the shear corresponding to the development of the probable moment capacity of the wall system at its plastic hinge locations, accounting for the magnification of shear forces due to inelastic effects of higher modes; or

b) the shear resulting from design load combinations which include earthquake with load effects calculated using $R_d R_o$ equal to 1.0.

The requirement (a) is based on a capacity design approach. This is to ensure that flexural yielding of the wall will occur prior to a shear failure. The requirement (b) is an upper limit to account for walls that may have inherent flexural strength well in excess of
that required, even with minimum reinforcement content. The seismic response of such walls will be essentially within the elastic domain. It is then unnecessary to design these walls for a shear demand greater than the elastic shear demand. For these cases, the capacity design approach is not needed. When walls are well-designed, the shear demand at probable flexural capacity required by (a) will generally be lower than the upper limit provided by (b) and therefore, will be the design shear demand. The following presents how the shear demand at probable flexural capacity, referred to as the capacity design or probable shear demand, is determined.

The CSA A23.3-04 does not specify any method to determine the probable shear demand and to account for the magnification of shear forces due to inelastic effects of higher modes. For the purpose of determining the probable shear demand, a simple approach is suggested in the CAC explanatory notes (CAC 2005) on the CSA A23.3-04 for ductile walls where plastic hinging is expected to form at the base of the walls. In this approach, the probable shear forces, $V_p$, for a wall are obtained by magnifying the factored shear forces, $V_f$, by the ratio $\gamma_p$ of the probable moment resistance, $M_{pw}$, of the wall section at plastic hinge to the factored moment, $M_f$, applied at the base of the wall. This can be expressed as:

$$V_p = \gamma_p V_f = \left( \frac{M_{pw}}{M_f} \right)_{base} V_f$$

(3.8)

The same approach can be used to determine the probable shear forces for the wall segments of ductile coupled walls. Both $M_f$ and $V_f$ in Equation 3.8 must account for elastic effects of higher modes. These effects are intrinsic to these parameters when their values are determined from a linear dynamic analysis method. It is noted that Equation (3.8) assumes that the ratio of the shear to moment at the base of a ductile wall remains constant as the moment increase to the probable resistance. However, inelastic dynamic analyses (Blakeley et al., 1975) showed that this ratio does not remain constant in multistorey wall buildings where higher vibration modes are dominant.

As in the 1984 and 1994 Canadian concrete design standards, the probable resistance in the CSA A23.3-04 corresponds to the expected strength for "ideal" construction with the steel exhibiting stresses above the specified yield strength $f_y$ (Mitchell and Paultre, 1994). This resistance takes into account the fact that the actual yield strength of reinforcing steel bars is usually greater than $f_y$ (by about 20% after Mirza and MacGregor (1979))
and the likely overstrength due to the development of strain hardening in the tension reinforcing bars. According to this definition, the probable moment resistance \( M_{puw} \) in Equation 3.8 must be calculated using an equivalent yield stress of \( 1.25f'_y \) in the tension reinforcing steel bars, the specified value of \( f'_c \), the resistance factors \( \phi_e \) and \( \phi_s \) equal to 1.0, and the appropriate axial force applied at the wall section of interest. For a coupled wall, this force must account for the earthquake-induced axial force, \( P_p \), resulting from the shear forces corresponding to development of probable moment resistance in the coupling beams above the wall section of interest. This results to a \( M_{puw} \) value greater for the "compression" wall than for the "tension" wall. For multistorey coupled-wall buildings where higher mode effects are significant, it is too conservative to assume that all coupling beams will develop their probable flexural resistance at the same instant in time. In these cases, it is more appropriate to use a reduced value of \( P_p \) to calculate \( M_{puw} \). The CSA A23.3-04 and its Commentary do not provide any specific method to reduce \( P_p \). Paulay (1986) recommends that the reduced value of \( P_p \) be estimated as follows:

\[
P_p^* = \left(1 - \frac{n}{80}\right) P_p
\]

(3.9)

where \( n \) is the number of floors above the wall section of interest, and its value should not be taken greater than 20.

Several analytical investigations (Blakeley et al., 1975; Iqbal and Derecho, 1980; Seneviratna and Krawinkler, 1994; Amaris, 2002) on the seismic behavior of ductile concrete cantilever wall systems showed that the higher vibration modes in long-period systems produce elastic seismic demands, particularly shear demand, that are much greater than those of the first mode and demand profiles that are significantly different from those of the first mode, over the entire height of the walls. These investigations also showed that the contribution of higher modes, particularly the second lateral mode contribution, to seismic strength demands greatly increases as the structural responses of such walls change from elastic to inelastic. Actually higher modes significantly amplify the seismic shear demand on a wall, particularly at wall base and at upper storeys, once a plastic hinge has formed at the base of the wall. The dynamic shear amplification at wall base was observed experimentally (Eberhard and Sozen, 1993). In tall cantilever wall structures, the dynamic amplification due to inelastic effects is such that flexural hinging may
form at storeys above the base of the wall under severe seismic events (Seneviratna and Krawinkler, 1994). Sangarayakul and Warnitchai (2004) explained that the increased higher mode effects in the inelastic responses of long-period wall buildings results from a significant increase in the second lateral mode contribution following a saturation of the first lateral mode response after the plastic hinge formation at wall base. The second lateral mode continues to respond elastically until its associated modal moment response reaches yielding level. This delayed change in the second mode response would be the cause of the significant increase in the second mode contribution.

Recent Canadian studies (Tremblay et al., 2001; Renaud, 2004) investigated the inelastic seismic response of ductile concrete shear walls of multistorey concrete buildings designed according to the NBCC 1995 and the CSA A23.3-94. The previous cited studies were performed using inelastic dynamic analyses and ground motion time histories compatible with the NBCC design ground motion. Results of these analyses show that the shear demand on cantilever walls may significantly be greater, particularly at wall base and upper storeys, than the capacity design shear demand determined with Equation (3.8). In addition, they indicate that the inelastic mechanism may not be constrained only at the base of cantilever walls, as presumed when the capacity design provisions of the CSA A23.3-94 are applied with the probable moment resistance of the walls matching or exceeding the assumed linear capacity moment demand after plastic hinge formation (see Fig. 3.4). It is of interest to note that no such predictions have been found in the available literature for ductile coupled walls of multistorey buildings designed according to the NBCC 1995 and the CSA A23.3-94, except when the selected input ground motions corresponded to seismic events that significantly exceeded the NBCC seismic design event (Renaud, 2004; White and Ventura, 2004). Moreover, a study performed by Chaallal and Gauthier (2000) from inelastic time-history dynamic analyses shows that the seismic design provisions in the CSA A23.3-94 provide a conservative shear design for ductile coupled walls of low- to mid-rise buildings (≤ 30 storeys), as long as the design shear demand calculated with Equation (3.8) is determined with the γp ratio of the "compression" wall. It appears then that the seismic design provisions of this standard conveniently accounts for the inelastic effects of higher modes for ductile coupled walls of low- to mid-rise buildings designed according to the NBCC 1995. It is noted that no such studies on the inelastic seismic responses of ductile cantilever or coupled walls of multistorey buildings designed according to the NBCC 2005 and the CSA A23.3-04 have been found in the literature.
currently available. In addition, no experimental validation of the aforementioned shear and hinging issues for such structures have been found in the literature. Nevertheless, the above results show that the inelastic effects of higher modes may cause issues for ductile walls, particularly for cantilever walls, of multistorey RC buildings designed in accordance with the seismic provisions of the current Canadian codes.

To account for these effects on shear forces in ductile concrete shear walls, the NZS 1995 recommends magnifying the shear forces at flexural capacity by a dynamic shear magnification factor, \( \omega_v \). This factor is equal to \( 0.9 + n/10 \) for buildings up to 6 storeys and \( 1.3 + n/30 \), to a maximum of 1.8, for taller buildings, where \( n \) is the number of storeys. The \( \omega_v \) values were derived from the values proposed by Blakeley et al. (1975), which were obtained from inelastic time-history dynamic analyses of isolated ductile concrete cantilever walls of multistorey buildings. It is important to note that the \( \omega_v \) values are adapted to the equivalent static force procedure of the New Zealand's building code. They assume that the base moment \( M_f \) in Equation (3.8) results from the code-specified lateral base shear \( V_f \) when the latter is distributed over the height of the building according to the code distribution, which is primarily the shape of the first flexural mode of vibration of the building. Moreover, the \( \omega_v \) factor does not account for the ground motion acceleration intensity, though dynamic shear magnification strongly depends on this parameter. This may result in non-conservative design even for seismic events at design level (Amaris, 2002). Although an adaptation of this dynamic shear magnification factor to Canadian design codes is recommended in the CAC 1995, no such recommendation is given in the CAC 2005. Actually the latter does not provide any suggestion on how to account for the dynamic magnification of shear forces in ductile concrete walls due to inelastic effects of higher modes. Although other methods (Iqbal and Derecho, 1980; Kabeyasawa and Ogata, 1985; Ghosh and Markevicius, 1990; Keintzel, 1990; Priestley, 2003) than the New Zealand concrete design standard's method have been proposed to account for this magnification effect in ductile concrete walls, very few methods (Filiatrault et al., 1994) have been proposed for the Canadian design codes. No such method specific to the new Canadian seismic code provisions is available at this time.

It is of interest to note that results from the various analytical studies presented in this section were obtained from inelastic time-history dynamic analyses. In such analyses, shear is generally more sensitive to higher mode responses and varies more rapidly with
time relative to the accompanying moment. This sensitivity coupled to deficient structural models can easily result to an overestimation of the shear demand on walls. The modelling issue is briefly discussed in Section 4.2 of this document. Other important parameters, such as the dynamic characteristics of the input motions used for analysis, may also explain sometimes the conservatism or non-conservatism of the analysis results presented in this section. For instance, Kabeyasawa and Ogata (1985) found that the dynamic fluctuation of higher-mode shear along the wall height is quite similar to the input ground acceleration waveform. Therefore, prior to using the analysis results presented in this section to modify the Canadian seismic code provisions, those should be validated with laboratory tests simulating adequately the inelastic seismic response of ductile concrete shear walls designed according to these provisions.
Chapter 4

Inelastic Seismic Analysis of Ductile

RC Walls

The default method of analysis in the NBCC 2005 for determining the design earthquake actions in building structures is the dynamic analysis. The NBCC permits the use of either linear (elastic) dynamic methods, such as the modal response spectrum method, or an inelastic time-history dynamic method. For reinforced concrete (RC) structures, the detailing configuration in members is usually unknown before any analysis is carried out. Consequently, the linear methods are generally used for design as the structural models associated to these methods simply require an estimation of the effective stiffness properties of members. However, such models cannot account for the inelastic effects of higher modes on the seismic response of a structure designed to dissipate inelastic energy. As discussed in Section 3.2.3 of this document, these effects can be very significant for long-period structures such as multistorey ductile cantilever walls. Therefore, the inelastic time-history dynamic method is generally used to verify a seismic design or to assess the seismic performance of a design previously established from linear dynamic methods. For these purposes, an inelastic static (pushover) method may also be useful.
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If the inelastic time-history dynamic method is used for seismic design, the NBCC 2005 Commentary provides recommendations regarding the ground motion time histories used as input motions and the inelastic structural modelling of the structure being design. The recommendations for the input motions are presented in Section 3.1.5 of this document. Those related to structural modelling refer to the current good practices for inelastic modelling. These practices are presented in this chapter. It is noted that the importance factor $I_E$ of the building (see Eq. (3.2)) must be accounted for in response history analysis either by scaling the design ground-motion time histories or by reducing the acceptable deflection and ductility capacities (Saatcioglu and Humar, 2003).

In this chapter, the basic concepts of the methods of inelastic analysis are briefly outlined as well as their pros and cons for seismic analysis. In addition, the most common structural modelling approaches for inelastic seismic analysis of ductile concrete shear walls are presented. The modelling approaches related to foundation, soil and soil-structure interaction are not considered in this chapter, as earthquake excitations are assumed to be imposed at the base of the walls. Foundations of the walls are assumed to be adequate to transmit earthquake actions from the walls into the ground without allowing the walls to rock.

4.1 Methods of Inelastic Seismic Analysis

As presented previously, the pushover analysis and the inelastic time-history dynamic analysis are the methods of inelastic analysis used in earthquake engineering. The purpose of the pushover analysis is to assess the expected performance of a structural system by estimating its strength and deformation demands in design earthquakes and comparing these demands to available capacities at the performance levels of interest (Krawinkler and Seneviratna, 1998). This method allows to verify the completeness and adequacy of the load path and to expose design weaknesses, such as storey mechanisms and excessive deformation demands. A pushover analysis is an incremental-iterative solution of the static equilibrium equations. It consists in applying a monotonically increasing static lateral load pattern, which is either a displacement or force pattern, to the structure. The loading is steadily increased either to a predetermined level of base shear, top displacement
or to a collapse of the structure (formation of an inelastic mechanism), whichever occurs first.

In the conventional pushover analysis, the lateral load pattern is kept constant during the analysis. This load pattern is often the code-specified force distribution. However, several studies (Mitchell and Paultrte, 1994; Mwafy and Elnashai, 2001) showed that load-displacement curves obtained from a conventional pushover analysis depend on the load pattern used. This may lead to significantly different curves. In addition, the applicability of this method is essentially limited to regular and short-period structures that vibrate primarily in their first flexural mode of vibration. The method cannot represent adequately the inelastic effects of higher modes and therefore may not predict the actual deformation modes that may occur in a structure subjected to severe earthquakes. All these issues may give erroneous and non-conservative results, particularly for structures where higher modes are dominant (Krawinkler and Seneviratne, 1998; Mwafy and Elnashai, 2001). In order to improve the conventional method, adaptive pushover analysis methods (Elnashai, 2001) have been developed. In these methods, the static lateral load pattern changes during the analysis. The lateral force pattern used in some adaptive methods adjusts itself to the deformation of the structure. Load-displacement curves obtained from these methods are independent of the selection of the initial load pattern. Although the adaptive pushover analysis methods seem providing better demand predictions for building structures (Elnashai, 2001), still several issues need to be addressed. Some of the most important issues presented by Krawinkler and Seneviratna (1998) are the incorporation of torsional effects, the damage cumulation, the inelastic effects of higher modes and therefore the detection of all possible local mechanisms. In consequence, Krawinkler and Seneviratna suggest that a pushover analysis be complemented with other methods, such as the inelastic time-history dynamic analysis.

The inelastic time-history dynamic analysis is widely used to assess the seismic performance of a structure. The inelastic dynamic response of a structural system is obtained by integrating the dynamic equation of equilibrium of the system. Various direct time-integration methods (Kujawski et al., 1989) can be used for this purpose. Among these methods, the unconditionally stable implicit Newmark's constant average acceleration method (Newmark, 1959) is the most popular for seismic analysis. In spite of the numerical stability of this method, it is often suggested that the time step be less than
10% of the period of the highest mode of vibration that contributes to the response of the structure. A Newton-Raphson iteration within each time step is often used with this Newmark's method to reduce errors on results. It is important to note that the inelastic time-history dynamic analysis is relatively sensitive to the structural modelling, the input motion characteristics and the time-history integration scheme and parameters (time step...). The validity and the accuracy of the inelastic seismic responses predicted with this method are strongly dependent on these parameters. The influence of structural modelling on response is addressed in the following section.

4.2 Inelastic Structural Modelling for Ductile Walls

As computers are more powerful and structural analysis softwares are more user friendly, three-dimensional (3D) linear analysis is certainly the analysis approach the most widely used by structural engineers for seismic design. However, inelastic seismic analysis is generally performed with two-dimensional (2D) structural models due to the limited ability of the current technology to model and predict the 3D inelastic behavior of building structures. Therefore, this section addresses the 2D inelastic structural modelling of ductile concrete shear walls.

4.2.1 Stiffness Modelling

Member Modelling

In general, structural members of ductile concrete wall structures are modelled with beam-line finite elements as members are primarily subjected to flexural deformations. Wall members are modelled with beam or beam-column elements and coupling beams are modelled with beam elements. The beam-column element is generally used to model the wall members of coupled walls. These walls usually exhibit substantial changes in axial force during their response to earthquake motions due to coupling. The variations of the axial force in a wall member during response affect its strength, stiffness and
ductility. The most sophisticated beam-column elements are formulated to take into account the effects of change in axial force on wall strength and stiffness only (Takayanagi and Schnobrich, 1979; Saatcioglu et al., 1983; Keshavarzian and Schnobrich, 1985b). The application of these elements is limited to wall members whose axial forces are below the balanced point load level. In contrast, the simplest formulation of the beam-column element, which accounts only for the effect of change in axial force on flexural strength, does not have this limitation. In fact, due to this simplification, a simple yield surface approximating the axial force-flexural strength interaction diagram of the wall section can be used. It is noted that at least the axial force-flexural strength interaction effect should be considered in inelastic seismic analyses of coupled walls (Keshavarzian and Schnobrich, 1985a). This effect can affect significantly the maximum predicted shear forces and bending moments in wall segments of these structures (Saatcioglu et al., 1983; Keshavarzian and Schnobrich, 1985b). The variation of the inelastic axial stiffness of these walls should also be considered (Takayanagi and Schnobrich, 1979).

The inelastic behavior of the beam or beam-column line element, in general, follows the concept of either the one-component or general two-component model. The main reasons are that these models are computationally convenient and can be sufficiently accurate for many practical applications. The one-component model was proposed by Giberson (1967). It consists of a linearly elastic element with one equivalent inelastic rotational spring attached at each end, as shown in Figure 4.1. All the inelastic deformations of the modelled member are lumped at these two inelastic end springs, or zero-length (point) plastic hinges. By assuming a deformed shape of the member, the moment-rotation loading history at each hinge can uniquely and independently be specified by a hysteretic model. A double curvature is generally assumed as deformed shape. This assumption is relatively fair for beams and columns but may be incorrect for walls as they are exposed to a more general moment distribution than the asymmetric one. This is one of the main disadvantage of the one-component model. This model fixes the inflexion point in the member in spite of the fact that this point moves continually along the member during response. Due to this shift, the member-end rotations become a function of member-end loads at both ends. Moreover, the one-component model may be inappropriate for walls since the inelastic flexural behavior of these members can be expected to expand along the height of the member rather than being localized.
The first two-component model was developed by Clough et al. (1965). As shown in Figure 4.2, this model idealizes a member with two imaginary parallel elements: one elasto-perfectly plastic to consider yielding and the other fully elastic to represent strain hardening. The stiffness matrix of the member is simply the sum of the stiffnesses of these two elements. The main deficiency of this model is that it is only applicable to members that exhibit elasto-plastic behavior without any stiffness degradation. However, the original two-component model was extended by Takizawa (1976) into a general form, which allows simulating various inelastic behaviors through the use of appropriate hysteretic load-deformation models. The latter element model is referred to as the general two-component model. In a two-component model, the reduction of stiffness is considered over the entire length of the element while, in the one-component model, the reduction of stiffness is assumed to be localized at the two inelastic rotational end springs. While the one-component model is based on the assumption of three rotational spring in series, the general two-component model is based on two rotational springs in parallel.

The one-component and general two-component models cannot directly account for multidimensional phenomena as nonlinear strain distribution over the wall section, diagonal cracking, localized damage and inelastic shear-type deformations. In addition, the
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zero-length plastic hinge concept yields abstract representation of the actual nonlinearities. Appropriate hysteretic load-deformation models and associated hinge properties may be difficult to determine and may depend on the type of loading expected. To avoid these issues, alternative modelling approaches have been developed recently using as input parameters material constitutive laws rather than load-deformation relationships computed analytically by methods of mechanics. Some approaches (Bolander and Wight, 1991; El-Tawil et al., 2002) use plane stress and line finite elements to model the concrete part and the reinforcing bars of wall members, respectively. Other more sophisticated approaches use multilayered or fiber beam-line elements based on the classical beam theory (Euler-Bernoulli or Timoshenko) or on shear-resisting mechanisms (Martinelli, 2002). The sections of such elements are composed of a series of parallel layers (see Fig. 4.3) or fibers, and a sectional response is computed using the uniaxial material constitutive laws defined for each layer or fiber. Such modelling allows to account for axial force-flexure interaction and different phenomena relative to reinforced concrete members, such as concrete cracking and stiffening in tension, confinement effect in compression, cyclic behavior of concrete and steel, shear effects and opening and closing of cracks with recovery of stiffness. However, it may overestimate stiffness because the level of pre-cracking due to shrinkage and gravity loads is not explicitly modelled. It is noted that currently this modelling approach is limited to simple uniform members reinforced with reinforcing bars disposed along the longitudinal axis of the members.

One promising approach (Légeron et al., 2005; Kotronis and Mazars, 2005) for performance-based design (PBD) uses multilayered or fiber beam-line finite elements with uniaxial constitutive laws for concrete and steel based on continuum damage mechanics and plasticity theory, respectively. Using damage laws, such as the constitutive law of La Borderie (1991) for concrete, allows quantifying the levels of damage in tension and compression across the sections of members. In spite of this advantage, this modelling approach is based on a deterministic element formulation which does not account for spatial variability in the material properties, geometry of the structure or applied loads. To overcome these issues, stochastic fiber element formulations (Lee and Mosalam, 2004) have recently been developed. This modelling approach allows to perform the probabilistic assessment of RC structures and therefore, to conduct design on a probabilistic basis rather than on a deterministic basis. This is really advantageous for PBD.
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Although modelling approaches based on material constitutive laws are more rationale and usually more accurate than those based on load-deformation relationships, the computational effort involved and the expertise level required make them prohibitive at this time for general-purpose use.

![Multilayered beam-line finite element](image)

Figure 4.3: Multilayered beam-line finite element (Légeron et al., 2005)

**Hysteretic Models**

As indicated in the previous section, the one-component and general two-component element models require the use of hysteretic load-deformation models. For this purpose, several models have been proposed to represent the different hysteretic behaviors of reinforced concrete members. For RC cantilever wall members, it is important that the assumed hysteretic models be realistic since the inelastic effects of higher vibration modes are rather sensitive to the selected models (Priestley and Amaris, 2003).

To simulate the hysteretic response in flexure of reinforced concrete members subjected to earthquake motions, Otani and Sozen (1972) proposed the well-known modified Takeda model (see Fig. 4.4). This model accounts for stiffness degradation but not for strength decay. It is the most used model for reinforced concrete members due to its applicability over a wide range of hysteretic responses of such members. Unlike the original (trilinear) Takeda model (Takeda et al., 1970), the modified Takeda model is based on a bilinear primary curve. The first linear segment of this curve prior to yielding represents the effective (elastic) cracked flexural response. With such model, the uncracked elastic response cannot be accounted for. Unlike beams, this response can be very significant for tall walls due to the high axial compression from gravity loads, particularly at the base. To capture the uncracked elastic response of wall members, trilinear hysteretic models, such as the SINA model (Saiidi and Sozen, 1979) shown in Figure 4.4, are more appropriate. As the
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modified Takeda model, the SINA model accounts for stiffness degradation but not for strength decay. This model introduces a crack closing load ($M_{cc}$ in Fig. 4.4) to take into account pinching, which is typical of shear behavior. The SINA model is based on the original Takeda model. It is noted that the hysteretic models presented here do not account for the effects of a changing axial force during response. These models are applicable to members subjected to constant axial compression. Moreover, they are limited to the deformation range prior to strength decay. However, they can be used beyond this range if they are combined with explicit strength decay relationships.

Figure 4.4: Common hysteretic models for reinforced concrete members

Inelastic deformations in ductile shear walls generated during seismic response is not limited to flexure, even when the global behavior of the structure is governed by flexure. Laboratory tests (Oesterle et al., 1977) have indicated that the base hinging region of such walls does in fact exhibit behavior corresponding to yielding in shear under inelastic flexural deformation. These tests also have shown that shear yielding is triggered by flexural yielding, even if the base hinging region of the wall is designed to have a shear capacity greater than that corresponding to flexural yielding at capacity. This means that a capacity design does not ensure an elastic shear behavior while the wall deforms inelastically in flexure. For this reason, hysteretic models (Ozczebe and Saatcioglu, 1989; Hidalgo et al., 2002) were developed for shear response of reinforced concrete members subjected to shear force and bending moment reversals. Most of these models, however, do not account for changes in axial force during response. In spite of the existence of hysteretic
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shear models, shear behavior of members of ductile shear wall models is often assumed to be linearly elastic. Inelastic analysis results from Saatcioglu et al. (1983) suggest that this approach is reasonable to predict the inelastic dynamic response envelopes of ductile coupled walls under earthquake excitations. However, seismic analysis and test results from Shiu et al. (1984) show that the linear shear assumption is appropriate only for a heavily coupled wall system but not for a lightly coupled wall system. For the latter system, Shiu et al. demonstrated that the inelastic shear action in the walls has a significant effect on the response of the wall system, as the behavior of this system is governed by the individual walls. That is why, previously, Derecho et al. (1979) warned that the linear shear assumption may not be appropriate to predict seismic responses of ductile cantilever walls. In fact, Derecho et al. stated that the use in inelastic time-history dynamic analysis of ductile cantilever wall models that do not account for shear yielding may produce predictions in shear significantly greater than those that would be associated with models that account for shear yielding.

As observed from Figure 4.4, most hysteretic models consist of a primary curve and a set of hysteresis rules, within the primary curve, that define the branches of loading, unloading and reloading under reversed cyclic loading. These rules are empirical and are intended to simulate the actual cyclic member behavior, often as observed during tests. The primary curve is the envelope, the strength boundary, of hysteresis loops of load-deformation relationships. It is an idealization of the monotonic response up to the onset of strength decay of either the member behavior or the sectional behavior of the member. Depending on which behavior is represented, various analytical methods of mechanics can be used to compute the primary curve. Standard plane-section analysis is generally used to compute the sectional behavior of a member. To determine the member behavior, mathematical relations between the load and the deformation parameters must be established as well as a modelling of the plastic hinge region(s). Usually the primary curves of hysteretic flexural models used along with the one-component or general two-component element models idealize the sectional behavior of members. They are generally in the form of bending moment-curvature relationships. When such relationships are computed, it is important to incorporate strain hardening in steel and, if applicable, confinement effect in concrete.

Recently Adebar and Ibrahim (2002) proposed a simple trilinear bending moment-
curvature relationship to idealize the primary curves of slender reinforced concrete shear walls (with a percentage of vertical reinforcement, \( \rho_v \leq 3\% \)) subjected to earthquake motions. This relationship uses the concept of the upper-bound response and the lower-bound response. The former response corresponds to when a wall is loaded monotonically to failure without having been cracked in previous load cycles while the latter response is what it is observed when the wall is reloaded monotonically after having been severely cracked in previous load cycles. The latter response differ from the previous one by completely ignoring concrete tensile stresses. To define the trilinear bending moment-curvature relationship, four parameters are required. As shown in Figure 4.5, these parameters are the well-know gross section stiffness, \( E_c I_g \), cracked-section stiffness, \( E_c I_{cr} \) and flexural (yield) strength of the wall section, \( M_y \). The fourth parameter is a bending moment, \( M_t \), that defines the transition from the first to the second linear segments of the trilinear relationship. For the upper-bound response, \( M_t \) is due to both the tension-stiffening effect as well as the axial compression acting on the wall, and is significantly greater than the bending moment at first cracking, \( M_{cr} \). Adebar and Ibrahim proposed the following simple expression to determine \( M_t \):

\[
M_t = \beta_u f_{cr} S + P \left( \frac{S}{A_g} + 0.08 \ell_w \right)
\]

where \( \beta_u \) equals to 0.0 or 1.5 for the lower- or upper-bound response, respectively; \( f_{cr} = 0.3(f'_c)^{1/2} \) (in MPa units) is the tensile cracking strength of concrete; \( S \) is the elastic section modulus \((I/c)\) to the tension face of the wall; \( P \) is the applied axial compression, and; \( A_g \) and \( \ell_w \) are the gross cross-sectional area and the horizontal length of the wall, respectively.
Adebar and Ibrahim validated their proposed trilinear relationship with static and cyclic pushover tests on a large-scale slender reinforced concrete wall. It is noted that the trilinear relationship as proposed by Adebar and Ibrahim does not account for the effect of strain hardening in steel. This effect can easily be implemented in the relationship by introducing the ultimate flexural resistance of the wall section, $M_u$, and its associated curvature, $\phi_u$, which can readily be determined from a standard plane-section analysis.

The work from Adebar and Ibrahim (2002) shows the importance of accounting for the uncracked flexural stiffness when simulating the inelastic flexural behavior of reinforced concrete shear walls. Also it points out, as other works (Ayoub and Filippou, 1998; Kwak and Kim, 2001), the importance of the tension-stiffening effect on the flexural stiffness of such walls, even after the concrete is severely cracked. Experimental investigations (Williams, 1986) have shown that this effect is mostly dependent on the diameter of reinforcement bars, reinforcement ratio, the distribution of reinforcement and shrinkage, especially for heavily reinforced members. According to Kaklauskas and Ghaboussi (2001), the tension-stiffening effect is particularly significant for members with low reinforcement ratios and small reinforcement diameters. This might explain the importance of the tension-stiffening effect in reinforced concrete shear walls. However, previous findings are mostly based on pushover (static) analysis and test results. Different works (Vecchio, 1999) showed that the tension-stiffening effect diminishes significantly as a result of extensive cyclic loading beyond yielding. For this reason, tension-stiffening effect is often neglected for seismic analysis of ductile concrete shear walls as considerable inelastic flexural deformations are expected in these structures. Nevertheless, previous findings suggest that tension stiffening of concrete be taken into account in the inelastic modelling of reinforced concrete shear wall structures when no to low inelastic seismic deformations are expected. This is one of the recommendations made by Haselton and Deierlein (2005) for the modelling of reinforced concrete frame building elements with low reinforcing ratios.

The modified Takeda model is generally used to simulate the hysteretic response in flexure of coupling beams of reinforced concrete coupled walls subjected to earthquake motions. For this purpose, the modified Takeda model is usually in the form of a bending moment-curvature response and its bilinear primary curve represents often an elasto-perfectly plastic response for simplicity. Recent cyclic test results obtained by Adebar et al. (2001) suggest that this simple representation can idealize reasonably well the hysteretic
response envelope of actual slender coupling beams diagonally reinforced in accordance with the seismic provisions of the 1994 and 2004 edition of the Canadian concrete design standard. The effective elastic stiffness, which is the slope of the first line segment of the bilinear curve prior to yielding, can readily be determined with a plane-section analysis or by using the appropriate effective section properties presented in Table 3.4 of this document. As shown in Figure 4.4, inelastic stiffness during unloading and reloading can be controlled in the modified Takeda model with the parameters $\alpha$ and $\beta_r$, respectively. These parameters are difficult to determine without test results. However, various studies (Saatcioglu et al., 1983; Chaallal and Gauthier, 2000) showed that variations in these parameters, within the range observed in tests, do not significantly affect dynamic response. Typical values of $\alpha$ and $\beta_r$ lie between 0.0 and 0.6 and 0.0 and 0.5, respectively. Coupling beam modelling should account for strength decay if the beam ductility demand reaches the range where the expected strength loss should occur (Saatcioglu et al., 1983). Recent test results (Adebar et al., 2001) suggest that no significant strength degradation prior to a curvature ductility ratio of about 8.0 should occur in slender coupling beams diagonally reinforced in accordance with the seismic provisions of the 1994 and 2004 edition of the Canadian concrete design standard.

It is of interest to note that analytical models (Paulay, 2001; Hindi and Hassan, 2004) of diagonally reinforced coupling beams can be used to determine the monotonic load-deformation relationships of hysteretic models more sophisticated than the modified Takeda model.

**Shear Wall Modelling**

Figure 4.6 presents the most common 2D structural modelling of reinforced concrete shear wall structures with beam-line elements. Cantilever walls are modelled with beam elements. Wall segments and coupling beams of coupled walls are modelled with beam or beam-column elements and beam elements, respectively. Usually the end regions of these elements can be represented with infinite stiffness in order to consider the finite widths of the adjoining members. This feature is important in the modelling of the connections between coupling beams and wall segments of coupled walls (see Fig. 4.6). The line elements representing the wall and beam members are generally located at the member centroids. It is typical to assume that earthquake-induced inertia forces at each floor of
the building are introduced to wall structures by diaphragm action of the floor system and by adequate connections to the diaphragm (Paulay and Priestley, 1992). This assumption means that the lateral displacements at each floor level of the wall segments of coupled wall models are the same. The contributions of the floor slab connected to coupling beams are generally neglected.

The inelastic element model and the finite element discretization of a member are two important parameters to consider when a structure is modelled. The most common inelastic element models used for beam-line elements are the one-component and general two-component models. A coupling beam is typically represented by a single one-component beam element, though the use of a single general two-component beam element would not affect seismic response results (Keshavarzian and Schnobrich, 1984). No such typical element configuration is observed for walls. The wall member of each storey is sometimes represented by means of a single one-component or general two-component finite element. However, Keshavarzian and Schnobrich (1984) showed that this modelling approach overestimates the maximum earthquake shear forces and bending moments at the base of walls. To get more accurate results, more than one element per storey may be used essentially over the expected base hinging region of a wall (see Fig. 4.7). This allows a better representation of the spreading of the inelastic flexural behavior along the hing-
ing region. However, when such modelling is considered, the sensitivity of the predicted responses to modelling should be investigated. A fine element discretization of a wall may produce a softening of the stiffness of the structural model. Few studies have investigated this issue with respect to various common inelastic beam element models.

4.2.2 Mass Modelling

For 2D dynamic analysis, the mass of each storey of a building is usually lumped at each floor level of the wall model, as it is normally assumed that the floors of a building will act as rigid diaphragms.

4.2.3 Damping Modelling

Damping is certainly one of the main issues in dynamic analysis. It represents the unknown nonlinear energy dissipation within a structure. Damping depends on various parameters, such as the material properties and geometrical characteristics of the members of a structure, and the level of excitation. Currently there is no method to determine damping from these parameters. Damping is normally assumed to be of the viscous type because of its mathematical simplicity.
In general, the viscous damping matrix used for dynamic analysis is assumed to be proportional to either the mass matrix, the stiffness matrix, or both. The latter model is referred to as Rayleigh damping model. This model has no physical meanings. Its use is purely governed by numerical reasons. In another numerically convenient viscous damping model, suggested by Wilson and Penzien (1972), the damping matrix is obtained by assigning a modal damping ratio, expressed as a percentage of critical damping, to each elastic mode of vibration of the structure. Modal damping ratio is also referred to as equivalent viscous damping ratio. Both modal and Rayleigh damping models are used in order to avoid the need to form a damping matrix based on the physical properties of the real structure.

The use of either previous viscous damping model requires values of modal damping ratios. A traditional value of 5% is normally used for elastic dynamic analysis of building structures subjected to earthquake motions. However, this value, for most structures, has very little experimental or theoretical justification (Wilson, 2002) and is assumed to account for the inelastic hysteretic energy dissipation. Typical values of modal damping ratios measured in actual undamaged mid- and high-rise reinforced concrete wall buildings lie between 1% and 2% (Boroschek and Yánez, 2000) and 0.5% and 1% (Brownjohn and Pan, 2001), respectively. Therefore, modal damping ratio values within these ranges should be used for the inelastic time-history dynamic analysis of such structures. It is important that appropriate damping values be used in order to not considerably underestimate or overestimate the seismic demand on a structure.
Chapter 5

Project Objectives and Methodology

This chapter presents a summary of some important issues outlined in the previous chapters of this document, the objectives of the present research project and the methodology adopted to achieve these objectives.

5.1 Summary of Important Issues

The literature review presented in the previous chapters of this document outlined several issues related to the performance-based seismic design, the seismic provisions of the NBCC 2005 and CSA A23.3-04, the seismic design of ductile concrete shear wall structures designed according to these codes and the inelastic seismic analysis of such structures. Some of the important issues are the following:

- The seismic provisions of the NBCC 1995 and the CSA A23.3-94 for ductile concrete shear walls, particularly ductile cantilever walls, do not appear suitable to account for inelastic effects of higher modes that occur in multistory ductile concrete wall
buildings. Actually the seismic shear demand on shear walls may significantly be greater than the seismic design shear forces, especially near the base of the walls, and the inelastic mechanism may not be constrained only at the base of cantilever walls, as presumed in the CSA A23.3-94;

- No experimental validation of the previous issues has been found in the literature currently available;

- The seismic provisions of the NBCC 2005 do not take into account the magnification of shear forces in building structures due to inelastic effects of higher modes. Moreover, no method is prescribed in the CSA A23.3-04 or recommended in the CAC 2005 to account for this dynamic magnification in ductile concrete shear walls, though the CSA A23.3-04 requires to account for it.

- No study verifying or assessing from inelastic analyses the seismic design or performance of multistorey ductile concrete shear walls designed according to the NBCC 2005 and the CSA A23.3-04 has been found in the literature currently available;

5.2 Project Objectives

From the issues outlined in the previous section, the main objective of this research project is to assess from inelastic analyses the seismic performance of a ductile concrete shear wall system used as a Seismic Force Resisting System (SFRS) for a multistorey concrete building designed according to the NBCC 2005 and the CSA A23.3-04. The secondary objectives of this project are the following:

1. To design and detail all structural components of the earthquake-resisting concrete wall building according to the NBCC 2005 and the CSA A23.3-04;

2. To numerically model the inelastic seismic behavior of the SFRS using the current state-of-the-art modelling technics for two-dimensional (2D) inelastic seismic analysis;
3. To verify the seismic design and to assess the seismic performance of the SFRS from inelastic static (pushover) and time-history dynamic analyses;

5.3 Methodology

The methodology adopted to achieve the objectives outlined in the previous section is as follows:

1. Defining the structural configuration of the multistorey reinforced concrete wall building studied in this research project, considering that the SFRS is a cantilever wall in one direction and a coupled wall in the orthogonal direction;

2. Designing and detailing all structural components of the earthquake-resisting concrete wall building according to the NBCC 2005 and the CSA A23.3-04, considering that the SFRS is ductile, as defined in the NBCC 2005;

3. Modelling numerically the inelastic seismic behavior of the SFRS using the current state-of-the-art modelling technics for 2D inelastic seismic analysis;

4. Selecting earthquake ground motion histories compatible with the acceleration response spectrum used for seismic design; and

5. Computing the inelastic static (pushover) and time-history dynamic responses of the SFRS to selected earthquake loadings.

The following details each of the stages of the methodology adopted in this research project.

5.3.1 Defining the Studied Wall Building Structure

Prior to conducting any analysis, a building structure must be defined. Figure 5.1 shows an overview of the global structural configuration of the wall building studied in this
CHAPTER 5. PROJECT OBJECTIVES AND METHODOLOGY

project. This building is a 12-storey reinforced concrete office building and is assumed to be located in the Canadian city of Montréal and to be founded on soft rock. Its overall width, length and height are 30 m, 30 m and 48.65 m, respectively. The height of the first storey is 4.85 m while that of the higher storeys is 3.65 m. The structure of the building is made of normal density concrete with $f'_{c} = 30$ MPa and steel reinforcement with $f_{y} = 400$ MPa. It is made up of a centrally located elevator core wall, 12 floors, 32 columns per storey and 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 at the 13th floor level. Each floor consists of a 200 mm thick flat plate with 6 m spans. The cross-sectional dimensions of the columns are presented in Table 5.1. It is noted that each side column has a length of 400 mm along the exterior faces of the building. The cross-sectional dimensions of all spandrel beams are $400 \times 400$ mm. The core wall cross section measures 6 m by 8 m, center to center, and its thickness is 400 mm over the entire height of the wall. The core wall is composed of two C-shaped walls connected by two $400$ mm wide $\times$ 1 m deep $\times$ 1.8 m long coupling beams located at the ceiling level of each floor. This configuration results in a coupled wall in the East-West (E-W) direction and a cantilever wall in the North-South (N-S) direction. The core wall is defined as the SFRS of the building.

It is noted that the studied building structure is the same as that studied by Renaud (2004). However, in the previous work, the structure was designed according to the NBCC 1995 and the CSA A23.3-94 and coupling beams were $400$ mm wide $\times$ 600 mm deep $\times$ 2 m long. These dimensions result to a less heavily coupled wall system. The work of Renaud (2004) is one of the recent studies that highlight the possible issues with Canadian seismic code provisions to underestimate the shear force demand at design level on ductile concrete shear wall systems.

5.3.2 Designing and Detailing the Wall Building Structure

The building structure previously defined for this research project is designed and detailed according to the NBCC 2005 and the CSA A23.3-04, considering that the earthquake-resisting core wall is ductile, as defined by the NBCC 2005. The design and detailing of
Table 5.1: Cross-sectional dimensions (in mm) of the columns of the studied building

<table>
<thead>
<tr>
<th>Type of column</th>
<th>Storeys</th>
<th>Width</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corner column</td>
<td>All</td>
<td>350</td>
<td>350</td>
</tr>
<tr>
<td>Side column</td>
<td>1 to 6</td>
<td>400</td>
<td>400</td>
</tr>
<tr>
<td></td>
<td>7 to 9</td>
<td>350</td>
<td>400</td>
</tr>
<tr>
<td></td>
<td>10 to 12</td>
<td>300</td>
<td>400</td>
</tr>
<tr>
<td>Interior column</td>
<td>1 to 3</td>
<td>500</td>
<td>500</td>
</tr>
<tr>
<td></td>
<td>4 to 6</td>
<td>450</td>
<td>450</td>
</tr>
<tr>
<td></td>
<td>7 to 9</td>
<td>400</td>
<td>400</td>
</tr>
<tr>
<td></td>
<td>10 to 12</td>
<td>350</td>
<td>350</td>
</tr>
</tbody>
</table>

the building structure is presented in details in Chapter 6 of this document.

5.3.3 Modelling Inelastic Seismic Wall Behavior

The inelastic seismic behavior of the SFRS is modelled using the current state-of-the-art modelling technics for 2D inelastic seismic analysis. Structural wall models are developed with the 2D finite-element (FE) structural analysis program RUAUMOKO (Carr, 2002b). A modelling assessment is conducted in order to ensure realistic and reliable numerical predictions. The 2D FE code EFICoS (Ghavamian and Mazars, 1998) is used for this assessment. Chapter 7 of this document presents in details the modelling assessment performed and the inelastic structural models developed for this project.

5.3.4 Selecting Earthquake Ground Motion Histories

Earthquake ground motion histories are required to carry out inelastic time-history dynamic analysis. To verify the seismic design of a structure with such analysis, the NBCC 2005 requires that the ground motion time histories used as input motions be compatible (see Section 3.1.5 of the present document) with the acceleration spectrum used for seismic
Table 5.2: Simulated NBCC 2005-compatible ground motion records selected and tuned for the city of Montréal

<table>
<thead>
<tr>
<th>Eastern Canadian Records (ECR) Selected from Atkinson (1999)</th>
<th>Tuned ECR for Montréal</th>
</tr>
</thead>
<tbody>
<tr>
<td>M</td>
<td>R</td>
</tr>
<tr>
<td>-----------------</td>
<td>-----</td>
</tr>
<tr>
<td>6.0</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>7.0</td>
<td>70</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

M: moment magnitude; R: epicentral distance

design. This spectrum is constructed from the design spectral acceleration values of $S(T)$ (Eq. (3.4) of this document) specified by the NBCC 2005 for the Canadian location and site condition of interest. It is noted that only horizontal ground motions are considered in this project.

Atkinson (1999) produced simulated ground-motion time histories, which are compatible with the uniform hazard spectra ($S_u(T)$) provided by the NBCC 2005 seismic hazard maps (probability level of 1/2500 per annum (p.a.)), for eastern and western Canadian locations. For the eastern Canadian locations, the simulated ground motion histories are produced for earthquakes of moment magnitude (M) 6.0 and 7.0 at various epicentral distances (R). According to Atkinson, the 1/2500 p.a. uniform hazard spectrum (UHS) for the Canadian city of Montréal can be matched by using the simulated records for eastern Canadian earthquakes of M6.0 at R=30 km (short-period events) and M7.0 at R=70 km (long-period events), and by scaling down their acceleration values with a fine-tune scale factor of 0.85 and 0.90, respectively. Both magnitude-distance combinations for this city must be applied to cover the entire frequency range of interest.

The ground motion records used as input motions in this project are selected from the suite of simulated NBCC 2005-compatible ground motion histories generated by Atkinson (1999) for eastern Canadian earthquakes of M6.0 at R=30 km and M7.0 at R=70 km, and
tuned with the appropriate fine-tune scale factor for the city of Montréal. Six simulated eastern records are selected, three for each magnitude-distance combination. These records are presented in Table 5.2. Their peak ground acceleration (PGA) value, total duration and duration of ground shaking are given in this table. It is noted that the M6.0 events have a shorter duration than the more distant M7.0 events. Table 5.2 also presents the PGA values of the selected records once tuned for Montréal, and the labels used in this project to identify these tuned records. The latter are plotted in Figures C.1 and C.2 in Appendix C of this document. Acceleration values of these records are uniformly time spaced at a time interval of 0.01 s. The 5% damped pseudo-absolute acceleration response spectra of these records are also plotted in Figures C.1 and C.2 and compared to the design UHS for Montréal. The latter spectrum is constructed from the $S_n(T)$ values, which are for a very dense soil or soft rock condition, specified by the NBCC 2005 for this city. This comparison shows, as expected, that the response spectra of the selected M6.0 and M7.0 events for Montréal match very well the design UHS for the short-period and long-period events, respectively. It is noted, however, that the response spectra are significantly lower than the design UHS for periods lower than 0.02 s and greater than 4.0 s.

As the building studied in this project is assumed to be founded on soft rock, the site coefficients $F_s$ and $F_v$ are equal to 1.0, as specified in the NBCC 2005. The selected ground motion records for Montréal then do not need to be scaled by these coefficients, as required by the NBCC to account for the amplification of seismic motions due to ground conditions. They do not need neither to be scaled by the importance factor $I_E$ of the building, as required by the NBCC 2005. Actually this factor is equal to 1.0 since the building is a normal office building. Therefore, the acceleration values of the simulated records used as input motions for the inelastic time-history dynamic analysis are those shown in Figures C.1 and C.2.

### 5.3.5 Computing Inelastic Seismic Responses

Inelastic seismic responses of the SFRS are obtained from 2D inelastic static (pushover) and time-history dynamic analyses. The computer program RUAAUMOKO is used to perform these analyses. As permitted by the NBCC 2005, independent analyses about
each of the horizontal principal axes of the SFRS are performed. These axes coincide with the N-S and E-W directions (see Fig. 5.1). The SFRS consists of a cantilever wall in the N-S direction and a coupled wall in the E-W direction. It is noted that torsion and sway ($P - \Delta$) effects are not considered for inelastic analysis. Preliminary analyses have shown that sway effects can be ignored as they have a negligible influence on computed responses.

The method of analysis used to compute the inelastic static responses is the adaptive pushover analysis method. This method ensures that computed responses are independent of the selection of the initial lateral load pattern applied over the height of the analyzed structure. The initial lateral load patterns used in this project are the earthquake-induced inertia force distributions determined for seismic design. The pushover analysis results are presented in Chapter 8 of this document in terms of:

- Base shear-overall drift responses of the wall for the factored, nominal and probable flexural resistances, as defined by the CSA A23.3-04;
- Sequences of hinge formation in the wall; and
- Damage states in the wall with respect to overall drift levels.

The inelastic time-history dynamic analyses are carried out using the implicit Newmark's constant average acceleration integration method with a constant time step of 0.001 s, and by using a Newton-Raphson iteration within each time step. An iteration tolerance small enough to minimize the numerical errors in predicted results is used. Preliminary analyses have shown that, for the desired level of damping, the use of time step values greater than 0.001 s produced sometimes numerical instabilities that stopped the computation, though the Newmark's constant average acceleration integration method is unconditionally stable.

In this project, a constant modal viscous damping ratio is assumed in all elastic modes of vibration of the analyzed structural system. Two modal viscous damping ratio values are used: 1% and 2% of critical. These values bound the range of typical modal viscous damping ratio values measured in actual undamaged mid-rise reinforced concrete wall buildings (Boroschek and Yánez, 2000).
Structural responses to earthquake ground motions are computed for a probable flexural resistance. The inelastic time-history dynamic analysis results are presented in Chapter 8 of this document in terms of:

- Overall drift, plastic hinges and associated curvature ductility ratios;
- Interstorey drift demand; and
- Flexural and shear demands on the wall structure;
Figure 5.1: Structural configuration of the studied 12-storey reinforced concrete building
Chapter 6

Design of a 12-Storey Ductile RC Wall Building

The earthquake-resisting structure of the studied 12-storey reinforced concrete (RC) wall building (see Fig. 5.1) is designed and detailed according to the NBCC 2005 and the CSA A23.3-04. The building is an office building and is assumed to be located in the Canadian city of Montréal and to be founded on soft rock. The core wall of the building is defined as the seismic force resisting system (SFRS) of the building. All other structural components of the building, i.e. slabs, spandrel beams and columns, are not considered as part of the SFRS. As seismic design is the main purpose of this project, the structure of the building is not design for wind load. It is assumed that the NBCC design load combinations considering wind load do not control the design of any structural components of the building. In addition, the design of foundations of the building is not considered in this project. It is assumed then that the core wall possesses an adequate foundation that can transmit earthquake actions from the structure into the ground without allowing the wall to rock.
CHAPTER 6. DESIGN OF A 12-STOREY DUCTILE RC WALL BUILDING

The core wall is designed and detailed to resist 100% of the earthquake loads and their effects, as required by the NBCC 2005, and to be ductile, as defined by this code, about each of the horizontal principal directions of the wall. The latter consists of a cantilever wall in the North-South (N-S) direction and a coupled wall in the East-West (E-W) direction. Diagonal reinforcement is provided in coupling beams to ensure a high level of ductility of the beams. Slabs are designed and detailed for gravity loads only. Spandrel beams and columns are designed and detailed to deform laterally with the building to the expected earthquake design displacement and to resist the earthquake-induced forces at this displacement as well as forces due to design gravity loads. In addition, columns are designed to preserve their vertical load carrying capacity while undergoing the earthquake design displacement.

6.1 Design Loading Cases

Table 6.1 presents the loading cases used for design, which is performed for ultimate limit states only. These loading cases are in accordance with the load combinations specified by the NBCC 2005 for ultimate limit states. For each loading case, principal loads are considered acting alone and with their companion loads, as required by the NBCC 2005. The following load combinations are considered for seismic design:

- \( 1.0D + 1.0E \)
- \( 1.0D + 1.0E + 0.5L + 0.25S \)

For the latter load combination, the companion-load factor 0.5 for live load \( L \) does not need to be increased to 1.0, as specified in the NBCC 2005, since the building is assumed not accommodating any storage occupancy, equipment area or service room in storeys above ground level.
Table 6.1: Design loading cases

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>Principal Loads</th>
<th>Companion Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$1.4D$</td>
<td>$S$</td>
</tr>
<tr>
<td>2</td>
<td>$1.25D + 1.5L$</td>
<td>$0.5S$</td>
</tr>
<tr>
<td>3</td>
<td>$1.25D + 1.5S$</td>
<td>$0.5L$</td>
</tr>
<tr>
<td>4</td>
<td>$1.0D + 1.0E$</td>
<td>$0.5L + 0.25S$</td>
</tr>
</tbody>
</table>

$D$: Dead load; $E$: Earthquake load; $L$: Live load; $S$: Snow Load

6.1.1 Gravity Loads

As presented in Table 6.1, gravity loads are the dead ($D$), live ($L$) and snow ($S$) loads. Table 6.2 gives the gravity load values used for design. These values are those specified in the NBCC 2005 and suggested in the Concrete Design Handbook 1995 (CDH 1995), except for the design snow load value. The latter value is determined from the NBCC 1995. The reason is that at the time the design of the building was performed, the ground snow and associated rain load values for the NBCC 2005 were not available. According to the NBCC 1995, the design snow load value for the present case is 2.32 kPa. As this value is for a 1-in-30 year return period, it must be increased to a value corresponding to a 1-in-50 year return period, which is the base return period adopted in the NBCC 2005 for snow load. A 10% increase has been used, as suggested in the NBC-95 Technical Changes document (NBCTC 2003) of the Canadian Commission on Building and Fire Codes. The resulting design snow load value is 2.55 kPa.

6.1.2 Earthquake Load

The earthquake design load $E$ is determined according to the Dynamic Analysis Procedure (DAP) of the NBCC 2005 using the linear modal response spectrum method as method of analysis. The Equivalent Static Force Procedure (ESFP) is not permitted by the NBCC 2005 since the height and the structural irregularity of the building, and the high seismic hazard level for Montréal. According to the NBCC 2005, the structure of the building is irregular as it is deemed torsionally sensitive (structural irregularity of Type 7). Actually
Table 6.2: Design gravity loads

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
<th>Source</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead loads ( (D) )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Self-weight of reinforced concrete</td>
<td>24.0 kN/m(^3)</td>
<td>CDH 1995</td>
<td>All structural members</td>
</tr>
<tr>
<td>Partitions</td>
<td>1.0 kPa</td>
<td>NBCC 2005</td>
<td>Over all floor areas</td>
</tr>
<tr>
<td>Ceiling and interior mechanical services</td>
<td>0.5 kPa</td>
<td>CDH 1995</td>
<td>Over all floor and roof areas</td>
</tr>
<tr>
<td>Exterior mechanical services</td>
<td>1.6 kPa</td>
<td>CDH 1995</td>
<td>Over roof area as Fig. 6.1 (a)</td>
</tr>
<tr>
<td>Roof insulation</td>
<td>0.5 kPa</td>
<td>CDH 1995</td>
<td>Over roof area</td>
</tr>
<tr>
<td>Exterior windows: 2 m high for storeys 2 to 12 and 3.2 m high for storey 1</td>
<td>0.38 kN/m(^2)</td>
<td>CDH 1995</td>
<td>For each building side, over storeys 1 to 12</td>
</tr>
<tr>
<td>Exterior facing wall: 1.65 m high</td>
<td>2.4 kN/m(^2)</td>
<td>CDH 1995</td>
<td>For each building side, over storeys 1 to 12</td>
</tr>
<tr>
<td>Live loads ( (L) ) due to use and occupancy</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Office area</td>
<td>2.4 kPa</td>
<td>NBCC 2005</td>
<td>Over all floor areas as Fig. 6.1 (b)</td>
</tr>
<tr>
<td>Corridor area around the core wall</td>
<td>4.8 kPa</td>
<td>NBCC 2005</td>
<td>Over all floor areas as Fig. 6.1 (b)</td>
</tr>
<tr>
<td>Snow load ( (S) )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Snow and rain</td>
<td>2.55 kPa</td>
<td>From NBCC 1995</td>
<td>Over all roof areas</td>
</tr>
</tbody>
</table>

its torsional sensitivity parameter \( B \), which is 1.86, is greater than 1.7.

The commercial structural analysis program ETABS was used to determine the earthquake design forces for the building. In this purpose, a three-dimensional (3D) elastic model of the isolated core wall was used since the core wall is designed to resist 100% of the earthquake loads and effects. This model is shown in Figure 6.2. Wall members were modelled with shell elements. Coupling beams, which have a span-to-depth ratio of 1.8, were modelled also with shell elements, as recommended in the ETABS User’s Guide for beams having a span-to-depth ratio lower than 3.0. Four shell elements per beam were used to represent adequately the flexural behavior of the beams. To make allowance for cracking, member stiffnesses were based on the effective section properties specified in the CSA A23.3-04 (see Section 3.2.1 of the present document). Wall members were
modelled with an effective moment of inertia of $0.7I_g$ and an effective axial area of $0.7A_g$, as determined with Equation (3.6) of this document. Coupling beams were modelled with an effective shear area of $0.45A_g$ and an effective moment of inertia of $0.25I_g$, as they were assumed to be diagonally reinforced. A modulus of elasticity of concrete, $E_c$, of 25000 MPa was assumed for the analysis. The 3D core wall model assumed that the floors of the building act as rigid diaphragms. Seismic mass of each storey was modelled as a lumped mass located at the center of mass of each floor level of the building. For the calculation of the seismic weights, which are equal to dead loads plus 25% of the design snow load, the partitions load was reduced to 0.5 kPa, as specified in the NBCC 2005. The total seismic weight, $W_s$, of the building is 86626 kN. The wall model was fully fixed at its base.

As permitted by the NBCC 2005, independent analyses about each of the horizontal principal axes of the SFRS were performed to determine the earthquake forces acting in the structure of the building. These principals axes are along the N-S and E-W directions (see Figure 5.1) and represent the earthquake design loading directions.

Using the NBCC empirical relation (see Equation (3.3) of the present document), the fundamental lateral period of vibration, $T_a$, of the 45-meter high RC wall building is 0.87 s. Using the 3D core wall model shown in Figure 6.2, the computed fundamental lateral period of vibration of the building is 1.41 s in the E-W direction and 1.74 s in
the N-S direction (see Table 6.3). As permitted by the NBCC 2005, $T_a$ values used for
design are those computed from the previous wall model. Both values are lower than or
equal to the upper limit specified by the NBCC 2005 for a RC shear wall structure when
$T_a$ is determined from a structural model. This limit is 1.74 s and is equal to twice the
fundamental lateral period calculated with the code-specified period formula (0.87 s).

Table 6.4 gives the NBCC 2005 earthquake design loads for the building. These loads
are expressed in terms of the shear force, overturning moment and accidental torsional
moment at the base of the building, for each earthquake design loading direction. They
were determined in accordance with the DAP of the NBCC 2005 using the linear modal
response spectrum analysis method. The DAP can be found in Section 3.1.5 of the present
document. The dynamic analysis was carried out with the 3D core wall model shown in
Figure 6.2. Structural elements not part of the SFRS were not accounted for in the
previous model as the added stiffness did not decrease the computed fundamental lateral
periods by more than 15%, as specified by the NBCC 2005. Figure 3.2 of this document
shows the constructed acceleration response spectrum for Montréal used for design. Values
of this spectrum do not need to be scaled by the site coefficients $F_A$ and $F_e$ since both coefficients are equal to 1.0 (the building is assumed founded on soft rock). The first three lateral modes were sufficient to get more than 90% of the participating mass of the structure for each earthquake design loading direction (see Table 6.3). Total lateral responses were obtained by combining the maximum modal responses of the considered modes with the SRSS method. The accidental torsional moments were obtained with the inertia force profiles determined from the dynamic analysis.

Table 6.4 gives the parameters used to calculate the minimum lateral earthquake force, $V$, at the base of the building (see Eq. (3.2) of this document). It is noted that the coupled wall in the E-W direction is considered fully coupled as its degree of coupling, which is 78.5%, is greater than 66%. The dynamic lateral earthquake force, $V_{\text{dyn}}$, is equal to the elastic base shear, $V_e$, determined from the dynamic analysis, times the ratio $I_e/R_dR_o$. As the structure of the building is designated as irregular, the design base shear, $V_d$, is the greatest of $V$ and $V_{\text{dyn}}$.

For comparison purposes, Table 6.4 gives the earthquake design loads for the $T_a$ values used for design, which are 1.41 s in the E-W direction and 1.74 s in the N-S direction, and those resulting from the use of the $T_a$ value calculated with the empirical equation specified in the NBCC 2005. The latter period value is 0.87 s. The use of this $T_a$ value for design would have increased by about 40% the earthquake design load values in the E-W direction and by about 44% those in the N-S direction. This is due in part to the
design spectrum shape for Montréal. As shown in Figure 6.3, design spectral response acceleration values increase by about 90% and 170% as period values drop to 0.87 s from 1.41 s and 1.74 s, respectively. This shows that the use of computed $T_a$ periods, which tend to be longer than those calculated with the NBCC empirical equations, may reduce significantly earthquake design loads and therefore building costs as the sizes of members and the amount of reinforcement required may be less.

It is of interest to note that the design base shear $V_d$ in the N-S direction (cantilever wall direction) is mainly due to the second lateral mode response, as illustrated in Figure 6.4(a). This figure also shows that the seismic shear force profile along the height of the building is mostly governed by the second lateral mode response, particularly at the base of the building and at upper storeys. Figure 6.4(b) indicates that the second lateral mode response also has a strong influence on the seismic overturning moment profile along the height of the building. However, this time, the influence of the second lateral mode is more significant at upper storeys than at the base of the building. Actually the base overturning moment is mostly governed by the first lateral mode response. Above the base, the overturning moment profile along the height of the building is controlled by the second lateral mode response. Figure 6.5 shows that all previous observations also apply in the E-W direction (coupled wall direction). However, in this direction, the influence of the second lateral mode response is less significant, particularly for the overturning moment at the base of the building. All above observations indicate that higher mode effects play a major role on the seismic forces applied to the studied wall building structure, especially
in the N-S direction. This is mainly due to the long fundamental vibration periods of the wall system and the high level of earthquake severity (2500-year return period) used for design.

6.2 Seismic Lateral Deformations

Table 6.5 gives the anticipated lateral drifts of the building deformed at seismic design displacements. These drifts were obtained from the 3D core wall model shown in Figure 6.2 and account for the inelastic, torsional and P-delta effects, though the latter effects are negligible. They mainly result from the first lateral mode response of each earthquake design loading direction. As shown in Table 6.5, all anticipated drift values are lower than 0.30%. This is considerably less than the drift limit of 2.5% specified by the NBCC 2005 for this office building. Therefore, the structure of the building complies with the NBCC drift requirements.

6.3 Detailing of Building Components

The detailing of all structural components of the building can be found in Appendix A of this document. These components are the slabs, the spandrel beams, the columns and the core wall. The detailing provided to all structural components is in accordance with the CSA A23.3-04. Since this project is about the seismic behavior of a ductile wall, a close-up on the detailing of the core wall is performed in this section.

It is noted that no special detailing for plastic hinges is provided to structural members not considered as part of the SFRS, though the CSA A23.3-04 requires such detailing if specific requirements are not meet. For these members, standard detailing has shown in general to be more than sufficient to resist design gravity forces and forces induced in the members when the building is deformed laterally to the earthquake design displacement. Although earthquake-induced forces were not obtained from an inelastic analysis, as rec-
ommended in the CAC 2005, the great conservative resistance of most members allows to assume that their detailing is appropriate.

The core wall is detailed according to the special provisions in the CSA A23.3-04 for seismic design of ductile shear walls. The detailing of this structure is based on the inelastic mechanisms shown in Figure 3.3 of this document for the E-W (coupled wall) and N-S (cantilever wall) directions. The following outlines the detailing characteristics of the core wall:

1. The first three storeys, from the wall base, of wall members are detailed as a plastic hinge region. This height is governed by the wall length \( \ell_w \) in the N-S direction and is slightly higher than the required minimum height of \( 1.5\ell_w \);

2. The flexural (vertical) reinforcement in wall members is governed by the required minimum reinforcement over the entire height of the wall system. Flexural reinforcement is composed of distributed and concentrated vertical reinforcements, as shown in Figures A.5 and A.6;

3. The concentrated vertical reinforcement in wall members changes from 25M to 20M bars above storey 6. This change is governed by the flexural strength of the cantilever wall system. It is based on the probable moment resistance of a C-shaped wall matching or exceeding the assumed linear probable moment demand after plastic hinge formation, as illustrated in Figure 6.6(a);

4. The shear (horizontal) reinforcement in the assumed base plastic hinge region of wall members is governed by the shear strength required to develop the probable flexural capacity (\( V_p \), Eq. (3.8)) of a C-shaped wall in the N-S direction. Above storey 4, the shear reinforcement in wall members is governed by the required minimum reinforcement, as shown in Figure 6.6(b);

5. Diagonal reinforcement is provided in coupling beams, as shown in Figure A.4. The beam reinforcement yielding strength matches the beam shear demand, after redistribution, as closely as possible.

It is noted that the proportion of concentrated vertical reinforcement at the web-flange intersections of C-shaped walls could be less than that used in this project, though
the latter proportion is governed by the required minimum reinforcement. In fact, for flanged walls, the CSA A23.3-04 permits now that concentrated vertical reinforcement at the ends of the effective flanges supply up to one half of the required minimum wall web concentrated reinforcement with the remaining placed at the end of the wall web. For the present case, this detailing approach would result to less concentrated vertical reinforcement in the core wall and therefore to a slightly lower but still conservative flexural resistance of the wall system. This approach has not been adopted in this project since the associated Clause in the CSA A23.3-04 was not included in the draft version of this Standard used for design.

As previously indicated, the linear probable moment demand, suggested in the CAC 1995, was used for the flexural design of wall members, even if the CSA A23.3-04 now prescribes a capacity moment demand for this purpose. The reason is to study whether in the present case the linear probable moment demand can prevent, unlike reported elsewhere (Tremblay et al., 2001; Renaud, 2004), the formation of plastic hinges above the wall base at design level. It is of interest to note that the flexural strength of wall members between storeys 6 and 10 would have been significantly greater if the capacity moment demand prescribed by the CSA A23.3-04 had been used for design. As shown in Figure 6.6(a), this demand at these storeys is significantly greater than the provided factored moment resistance, \( M_r \), resulting from the application of the linear probable moment demand. Nevertheless, \( M_r \) is more than conservative compared to the NBCC factored moments (\( M_f \)).

Table 6.6 gives the base flexural and shear resistances of the core wall under design loads. Maximum and minimum moment resistances are determined for axial loads corresponding to seismic loading cases 1.0\( E \) + 1.0\( D \) + 0.5\( L \) + 0.25\( S \) and 1.0\( E \) + 1.0\( D \), respectively. Table 6.6 also gives the wall overstrength factor, \( \gamma_w \), and the wall overstrength factor at probable flexural capacity, \( \gamma_p \), for each earthquake design loading direction. The former factor is determined for a wall system, as defined in the CSA A23.3-04, while the latter is determined for an individual wall. This difference is due to the fact that \( \gamma_p \) is used to determine from Equation (3.8) the probable shear demand on the individual C-shaped walls and not on the wall systems.

From Table 6.6, \( \gamma_w \) is about 3.6 while \( \gamma_p \) is lightly greater than 4.0, for both principal flexural directions. These relatively great wall overstrength values are due to low NBCC
factored moments ($M_f$) in comparison with the flexural resistances of the wall members, as illustrated in Figure 6.6(a) for the cantilever wall system. If earthquake design loads were based on the empirical $T_a$ value of 0.87 s (see Section 6.1.2), $M_f$ values would be approximately 40% greater and then $\gamma_w$ and $\gamma_p$ values would be reduced to about 2.5 and 3.0, respectively. Moreover, lower $\gamma_w$ and $\gamma_p$ values could be expected if the base flexural reinforcement of wall members was governed by $M_f$, rather than by the required minimum reinforcement, since $M_r$ could be set closer to $M_f$. However, $\gamma_w$ should not be less than 1.3, as specified in the CSA A23.3-04. This shows that the values of the wall overstrength factors are influenced by the $T_a$ value(s) selected for design. For the designed core wall, the selected $T_a$ values, which are computed values, produced relatively great $\gamma_w$ and $\gamma_p$ values. This should produce low anticipated inelastic rotational demands but great probable shear demands on the wall. It is noted that there is no upper limit specified in the CSA A23.3-04 for $\gamma_w$.

Although it was not performed in this project, two different approaches could be used to ensure that the earthquake design moments, $M_f$, control the base flexural reinforcement of the wall members. The first approach would be to reduce the sizes of the wall members and the second one would be to reduce the level of expected ductility, that is, designing and detailing for $R_d < 3.0$. Both approaches also could be applied simultaneously. The first approach is obviously the preferred one if the level of desired ductility is maintained to $R_d > 3.0$.

The design base shears for a C-shaped wall are given in Table 6.6. They correspond to the development of the probable moment capacity of the wall members at their base. These probable base shears ($V_p$) were calculated with Equation (3.8) of this document, using the NBCC factored forces ($M_f$ and $V_f$) determined from the SRSS combination. Despite the relatively great flexural overstrength of the core wall, the calculated probable base shears are less than the elastic base shears calculated using $R_d R_o$ equal to 1.0. Their values are about 35% and 26% lower than the elastic base shear values ($R_d R_o = 1.0$) in the E-W and N-S directions, respectively. Nevertheless, the design base shear for a C-shaped wall in the N-S direction is equal to the upper limit specified in the CSA A23.3-04 for the factored shear resistance in a region of plastic hinging. This shows that excessive flexural overstrength may produce a shear demand that exceeds the maximum shear capacity of a wall and therefore, may be detrimental rather than beneficial for a
wall. It is important to point out that the probable shear demand in the N-S direction calculated with Equation (3.8) is independent of the $T_a$ value used for design since the ratio $V_f/(M_f)_{base}$ in this equation does not change with $T_a$. This means that the same N-S design (probable) shear demand would have been obtained using the $T_a$ value of 0.87 s calculated with the NBCC empirical relation, as shown in Figure 6.6(b).

It is noted that the design (probable) base shear in the E-W direction is that determined for the "compression" wall as the $\gamma_p$ value of this wall is greater than that of the "tension" wall (see Table 6.6). This design approach for a coupled wall should be conservative (Chaallal and Gauthier, 2000), though in the present case the $\gamma_p$ value of the compression wall is not significantly greater than that of the tension wall.

### 6.4 Ductility of the Wall System

Table 6.7 gives the anticipated inelastic rotational demands on the structural components of the core wall and their respective inelastic rotational capacities. These rotational demands and capacities were calculated according to the relations specified in the CSA A23.3-04. These relations can be found in Section 2.2.2 of the present document. Rotational values given in Table 6.7 show that the flexural ductility of the wall structure is sufficient since the inelastic rotational capacities are greater than the inelastic rotational demands.

It is noted in Table 6.7 the significantly low rotational demand values in comparison with the rotational capacity values. The inelastic rotational demands on the wall are even lower than the minimum inelastic rotational demand value of 0.004 rad. specified in the CSA A23.3-04 for a wall. This is mainly due to the low design lateral displacements (<100 mm) and the relatively great flexural overstrength of the wall structure (see Table 6.6). From these results, little inelastic deformations are expected in the wall structure, though this structure is designed and detailed to be ductile ($R_d > 3.0$).
6.5 Design Review

This chapter shows that the earthquake-resisting structure of the 12-storey ductile RC wall building complies with the NBCC 2005 and CSA A23.3-04 requirements. However, the following issues are pointed out:

1. The core wall structure has a relatively great flexural overstrength due to the excess strength arising from the required minimum reinforcement in the assumed base plastic region of the wall;

2. This flexural overstrength considerably increases the shear demand at flexural capacity on the wall members and produces relatively low inelastic deformations in the wall structure, though this structure is designed and detailed to be ductile ($R_d > 3.0$).

These issues are verified and investigated in this project through the use of inelastic seismic analyses of the core wall structure subjected to earthquake design loadings.
## Table 6.4: NBCC 2005 earthquake design loads for the 12-storey building

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Earthquake loading direction</th>
<th>E-W</th>
<th>N-S</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of SFRS</td>
<td>Ductile coupled wall</td>
<td>0.87</td>
<td>N/A</td>
</tr>
<tr>
<td>Degree of coupling</td>
<td>78.5% → fully</td>
<td>1.74</td>
<td>0.74</td>
</tr>
<tr>
<td>Fundamental lateral period, $T_a$ [seconds]</td>
<td>1.41</td>
<td>0.87</td>
<td>1.74</td>
</tr>
<tr>
<td>Acceleration-based or velocity-based site coefficient, $F_a$ or $F_v$</td>
<td>1.0 (Soft rock)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design spectral response acceleration, $S(T_a)$ [in fraction of g]</td>
<td>0.10</td>
<td>0.19</td>
<td>0.07</td>
</tr>
<tr>
<td>Factor to account for higher mode effects on base shear, $M_o$</td>
<td>1.04</td>
<td>1.0</td>
<td>1.74</td>
</tr>
<tr>
<td>Ductility-related force modification factor, $R_d$</td>
<td>4.0</td>
<td></td>
<td>3.5</td>
</tr>
<tr>
<td>Overstrength-related force modification factor, $R_o$</td>
<td>1.7</td>
<td></td>
<td>1.6</td>
</tr>
<tr>
<td>Earthquake importance factor of the building, $I_E$</td>
<td></td>
<td>1.0 (Normal)</td>
<td></td>
</tr>
<tr>
<td>Total seismic weight of the building, $W$ [kN]</td>
<td></td>
<td>86626</td>
<td></td>
</tr>
<tr>
<td>Minimum (static) lateral earthquake force at the base, $V$ [kN]</td>
<td>1355</td>
<td>2452</td>
<td>1938</td>
</tr>
<tr>
<td>Dynamic lateral earthquake force at the base, $V_{(d)}$ [kN]</td>
<td>1760</td>
<td>1760</td>
<td>2062</td>
</tr>
<tr>
<td>Design base shear, $V_d$ [kN]</td>
<td>1760</td>
<td>2452</td>
<td>2062</td>
</tr>
<tr>
<td>Design base overturning moment, $M_d$ [kNm]</td>
<td>32017</td>
<td>44604</td>
<td>29057</td>
</tr>
<tr>
<td>Design base accidental torsional moment, $T_d$ [kNm]</td>
<td>5351</td>
<td>7455</td>
<td>6269</td>
</tr>
<tr>
<td>Note: Bold force values are those used for design</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## Table 6.5: Anticipated lateral drifts at seismic design displacements

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Loading direction</th>
<th>NBCC limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global drift (at building roof, which is at 45 m high)</td>
<td>0.22%</td>
<td>2.5%</td>
</tr>
<tr>
<td>Maximum interstorey drift for all storeys</td>
<td>0.26%</td>
<td>2.5%</td>
</tr>
</tbody>
</table>
Figure 6.4: NBCC earthquake design forces in the N-S direction (cantilever wall direction) obtained from the SRSS combination method
Figure 6.5: NBCC earthquake design forces in the E-W direction (coupled wall direction) obtained from the SRSS combination method.
Figure 6.6: Capacity design demand on a C-shaped wall in the N-S direction
### Table 6.6: Base flexural and shear strengths of the core wall under design loads

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Earthquake loading direction</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>E-W (Coupled wall)</td>
<td>N-S</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Tension wall</td>
<td>Compression wall</td>
<td>Cantilever wall</td>
</tr>
<tr>
<td><strong>Flexural strength</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Factored base overturning moment on the wall system, $M_{fo}$ [kNm]</td>
<td>32017</td>
<td>29057</td>
<td></td>
</tr>
<tr>
<td>Design base moment per C-shaped wall, $M_f$ [kNm]</td>
<td>7120</td>
<td>7120</td>
<td>17500</td>
</tr>
<tr>
<td>Minimum factored moment resistance per C-shaped wall, $M_r$ [kNm]</td>
<td>22360</td>
<td>24100</td>
<td>56800</td>
</tr>
<tr>
<td>Minimum nominal moment resistance per C-shaped wall, $M_n$ [kNm]</td>
<td>26070</td>
<td>26500</td>
<td>62475</td>
</tr>
<tr>
<td>Maximum probable moment resistance per C-shaped wall, $M_{pw}$ [kNm]</td>
<td>28970</td>
<td>31290</td>
<td>72600</td>
</tr>
<tr>
<td>Wall overstrength factor (see notes below) of the wall system, $\gamma_w$ ($\geq 1.3$)</td>
<td>3.67</td>
<td></td>
<td>3.57</td>
</tr>
<tr>
<td>Wall overstrength factor, per C-shaped wall, at probable flexural capacity, $\gamma_p = M_{pw}/M_f$</td>
<td>4.07</td>
<td>4.39</td>
<td>4.15</td>
</tr>
<tr>
<td><strong>Shear strength</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design (probable) base shear per C-shaped wall, $V_p$ [kN]</td>
<td>3870</td>
<td></td>
<td>6300</td>
</tr>
<tr>
<td>Factored shear resistance per C-shaped wall, $V_r$ [kN]</td>
<td>4000</td>
<td></td>
<td>6300</td>
</tr>
<tr>
<td>Cantilever wall: $\gamma_w = M_n/M_f$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coupled wall: $\gamma_w = (M_n^{\text{tension}} + M_n^{\text{comp.}} + M_n^{\text{coupling}})/M_{fo}$</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

where $M_n^{\text{coupling}}$ is the nominal base moment resulting from coupling action and can be calculated as $\sum V_{bn} \cdot \ell_{cg}$ where $\sum V_{bn}$ is the sum of the shears corresponding to the nominal flexural resistance of coupling beams above the base of the wall, and $\ell_{cg}$ is the horizontal distance between centroids of walls on either side of coupling beams.
### Table 6.7: Inelastic rotational demands and capacities of the core wall components

<table>
<thead>
<tr>
<th>Description</th>
<th>Earthquake loading direction</th>
<th>Inelastic rot. capacity, $\theta_{ic}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>E-W</td>
<td>N-S</td>
</tr>
<tr>
<td>Type of ductile wall</td>
<td>coupled wall</td>
<td>cantilever wall</td>
</tr>
<tr>
<td>Lateral (elastic) deflection of the wall at building roof (45 m), $\Delta_f$ [mm]</td>
<td>12</td>
<td>16</td>
</tr>
<tr>
<td>Lateral yield deflection of the wall at building roof (45 m), $\Delta_y$ [mm] (See notes below)</td>
<td>24</td>
<td>57</td>
</tr>
<tr>
<td>Total lateral deflection of the wall at building roof (45 m), $\Delta_f R_d R_o$ [mm]</td>
<td>82</td>
<td>90</td>
</tr>
<tr>
<td>Inelastic rotational demand on the wall, $\theta_{id}$ ($\geq 0.004$) [radian]</td>
<td>0.004 (0.002)</td>
<td>0.004 (0.001)</td>
</tr>
<tr>
<td>Inelastic rotational demand on coupling beams, $\theta_{id}$ [radian]</td>
<td>0.007</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Cantilever wall: \[ \Delta_y = \gamma_w \Delta_f \]

Coupled wall: \[ \Delta_y = \gamma_w^2 \Delta_f / R_d R_o \] (Adebar et al., 2005)
Chapter 7

Inelastic Modelling of the Wall

Structure

This chapter presents the modelling adopted in this project to carry out two-dimensional (2D) inelastic seismic analyses of the SFRS, the core wall structure, of the multistorey concrete building designed in Chapter 6 of this document.

As indicated in Section 5.3.5, seismic analyses are performed about each of the horizontal principal axes of the core wall. Therefore, two independent and isolated wall models are developed: a cantilever wall model for the North-South (N-S) direction and a coupled wall model for the East-West (E-W) direction. Moreover, a 2D model of the whole building structure in the N-S direction is developed in order to take into account the stiffness of all structural components of the building. Actually the work of Renaud (2004), which was performed practically on the same core-wall building, indicates that the predicted maximum shear force and flexural demands on the cantilever wall structure may significantly differ when the added stiffness from the structural components not part of the wall system is accounted for in the inelastic time-history dynamic analysis. No such significative differences have been predicted in the coupled wall direction.
CHAPTER 7. INELASTIC MODELLING OF THE WALL STRUCTURE

Foundations of the building are not modelled as earthquake ground motions are imposed at the base of the structure. Foundations of the core wall are assumed to be adequate to transmit earthquake actions from the wall into the ground without allowing the wall to rock. Moreover, models do not take into account soil-structure interactions. In consequence, the wall and building models are fully fixed at ground level.

As indicated in Section 5.3.3, inelastic seismic analyses are carried out with the 2D finite-element (FE) structural analysis program RUAUMOKO (Carr, 2002b). This program has a wide variety of finite elements available to represent the members of the structural system and a large number of different hysteretic models available to represent the load-deformation relationships in these members. Moreover, it can perform many types of analysis and different associated computer programs, such as DYNAPLOT (Carr, 2002a), allow to post-process RUAUMOKO numerical results.

In order to get realistic and accurate numerical simulations from RUAUMOKO, a modelling assessment is conducted.

7.1 Modelling Assessment

As presented in Section 4.2, different structural modelling approaches can be used to simulate the inelastic seismic behavior of ductile concrete shear walls. The most common one represents wall members with inelastic beam-line finite elements to which are associated hysteretic models simulating the bending moment-curvature relationships in these members. This simplified modelling is computationally efficient and can produce relatively good numerical simulations when it is well done. It is the structural modelling approach adopted in this project for the numerical simulations with RUAUMOKO.

As pointed out throughout Section 4.2, inelastic structural modelling is a delicate issue. Several modelling parameters can influence the validity and the accuracy of the predicted responses. The review performed in Section 4.2.1 about stiffness modelling of ductile shear wall structures shows that few studies have investigated the influence of the following three structural modelling parameters on seismic response:
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- the inelastic beam element model: one-component vs general two-component model;
- the finite element discretization of wall members; and
- the tension stiffening of concrete.

In order to get reliable numerical results from the RUAUMOKO structural models developed for seismic analysis, an investigation on the previous parameters is conducted in this section.

Reference responses are needed for the investigation. Having no test data specific to the studied wall structure, these responses are determined from sophisticated numerical simulations. They are computed from the 2D FE code EFiCoS (Ghavamian and Mazars, 1998). This code is developed exclusively to simulate the inelastic behavior of reinforced concrete (RC) structures. It uses Bernoulli multilayered beam-line elements with uniaxial constitutive laws for concrete and steel based on continuum damage mechanics and plasticity theory, respectively. Previous works (Kotronis et al., 2005; Légeron et al., 2005) showed the ability of this code to accurately reproduce the global but also the local behavior, such as the opening and reclosure of cracks, of actual RC structures, even under severe dynamic loadings. However, the modelling approach used in EFiCoS has some limitations. It assumes linear shear, does not take into account bond slip and is inadequate to capture localized, discrete phenomena like excessive plastic deformations and significant cracks. Moreover, it is limited to simple uniform members reinforced with reinforcing bars disposed along the longitudinal axis of the members. This means that diagonally reinforced coupling beams cannot be directly modelled in EFiCoS. In addition, for this work, no post-processor was available to process EFiCoS numerical results. Therefore, in this project, responses predicted by EFiCoS are used only as a comparison basis to assess RUAUMOKO numerical results. Furthermore, this assessment is limited to the cantilever wall model and is based on a limited number of global structural response parameters.
7.1.1 RUAUMOKO Cantilever Wall Model

The whole cantilever wall structure is represented by a single-line stick model, as shown in Figure 4.6 of this document. The wall is modelled with inelastic beam-line finite elements, which are located at the wall centroid. The elastic material and section properties used in RUAUMOKO for these elements are given in Table B.1 in Appendix B of this document. The modelling of the inelastic behavior of the wall is limited to flexure. Its shear behavior is assumed to be linearly elastic. This is consistent with the modelling approach used in EFICoS.

In the purpose of the modelling investigation, the cantilever wall is modelled with two different common inelastic beam element models: one-component and general two-component models. Moreover, three different beam element discretizations of the wall are investigated. These are shown in Figure B.2. The first one, referred to as NSW1, considers a single element per storey. The second one, referred to as NSW2, is characterized by a fine discretization in the assumed base plastic hinge region of the wall and a coarse one above this region. The fine discretization allows a better representation of the spreading of the inelastic flexural behavior along the hinging region. Element length in this region is approximately 0.95 m. In the third one, referred to as NSW3, this element length is maintained over the entire height of the wall. Unlike NSW3, NSW1 and NSW2 are quite common beam element discretizations for a shear wall structure. However, NSW3 allows a better representation of the unexpected plastic hinges that may develop at upper storeys, as reported in some recent studies (Tremblay et al., 2001; Renaud, 2004) on ductile cantilever walls of multistorey concrete buildings designed according to the NBCC 1995 and the CSA A23.3-94. The RUAUMOKO cantilever wall model meshed according to the beam element discretization NSW1, NSW2 and NSW3 is referred to as NSW1-R, NSW2-R and NSW3-R, respectively.

It is noted that a plastic hinge length can be specified in RUAUMOKO for each plastic rotational end spring of a one-component beam element. No hinge length value is specified in the RUAUMOKO wall model. This makes the plastic curvature the same as the plastic hinge rotation. For this wall model, this is equivalent to specifying a plastic hinge length of 1.0 m at each element end.
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In order to investigate the concrete tension-stiffening effect on the inelastic seismic response of the wall, two limit cases are considered. In the first one, the concrete strength in tension is completely ignored in the computation of the wall response. This is generally assumed for inelastic seismic analysis. This case is referred to as the lower-bound (LB) case. In the second case, the concrete strength in tension, including the tension stiffening, is accounted for in the computation of the wall response. This case is referred to as the upper-bound (UB) case.

To study these two cases with the RUAMOKO wall model, the simple trilinear bending moment-curvature relationship proposed by Adebar and Ibrahim (2002) is used (see Fig. 4.5). This relationship, which idealizes the envelope of the hysteretic flexural behavior of a wall member, allows to account for, or not, the concrete strength in tension, including the tension-stiffening effect. Moreover, it captures the uncracked elastic response of the wall induced by the axial compression from gravity loads. The relationship is modified to account for strain hardening in steel. This is performed by including in the trilinear relationship the flexural ultimate strength of the wall section, $M_u$, and its associated curvature, $\phi_u$. From these parameters, the slope of the linear segment representing the yielding portion of the trilinear relationship can be determined.

Values of all parameters defining the trilinear bending moment-curvature relationship are determined with the sectional analysis program MNPHi (Paultre, 2001) using the material properties given in Tables B.2 and B.3, and their respective material stress-strain relationship shown in Figure B.1. The bending moment $M_t$ of the trilinear relationship is the only parameter that is not determined with MNPHi. Values of this parameter are calculated with Equation (4.1), which is the empirical equation proposed by Adebar and Ibrahim (2002). For the LB case, where the concrete strength in tension is completely neglected, the first part of this equation is omitted ($\beta_u = 0.0$). For this case, $M_t$, referred to as $M'_t$, is only due to the axial compression acting on the wall. The trilinear relationship associated to this case corresponds to the monotonic flexural response of a severely cracked wall member (see Fig. 4.5). For the UB case, where concrete tension-stiffening effect is accounted for, a $\beta_u$ value of 1.5 is used in Equation (4.1), as proposed by Adebar and Ibrahim (2002). For this case, $M_t$, referred to as $M''_t$, is due to both the tension-stiffening effect and the axial compression acting on the wall. The trilinear relationship associated to this case corresponds to the monotonic flexural response of a previously uncracked
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wall member (see Fig. 4.5). Trilinear relationships derived for the cantilever wall for the LB and UB cases have shown to be very good idealizations of the respective bending moment-curvature responses determined with MNPHi.

A trilinear bending moment-curvature relationship is defined for the wall member of each storey, as the axial compression acting on the wall decreases with increasing floor level. The only exception is for the wall member of storey 7. For this member, two trilinear bending moment-curvature relationships are defined: one for the lower half of the storey and a second one for the upper half. This is to account for the change of concentrated vertical reinforcement at this storey. Concentrated reinforcement changes from 25M to 20M bars above storey 6. It is assumed that this change is located over the lower half of storey 7 and the resulting lap splices are in accordance with the requirements of the CSA A23.3-04. This means that the concentrated reinforcement over the upper half of storey 7 and above is composed of 20M bars only. The trilinear bending moment-curvature relationship for the lower half of storey 7 is taken as the mean of the trilinear relationship for storey 6 and that for the upper half of storey 7.

To simulate the hysteretic response in flexure of the wall member of each storey, the trilinear hysteretic model SINA (Saïdi and Sozen, 1979) is used. As shown in Figure 4.4, this model accounts for stiffness degradation but not for strength decay. The crack closing bending moment $M_{cc}$ included in the model allows to take into account pinching, which is typical of shear behavior. In this project, $M_{cc}$ is taken as $M'$. The cracking bending moment $M_{cr}$ of the SINA model is taken as $M''$ for the LB case and $M'''$ for the UB case.

As indicated previously, the SINA hysteretic model does not account for strength decay. In RUAUMOKO, this issue is overcome by explicitly specifying a strength degradation relationship. Despite this feature, strength degradation is not accounted for in the RUAUMOKO wall model. This is in consequence of the design results presented in Section 6.4 of this document. Table 6.7 shows that low inelastic flexural deformations are expected. Actually the inelastic rotational demand on the cantilever wall is estimated to be about 0.001 radian, which corresponds for this wall to a displacement ductility ratio, $\mu_d$, of about 1.6 ($= R_d R_d / \gamma_w$). For such walls, no significant strength degradation can be expected up to about $\mu_d = 3.0$ (Riva et al., 2003). Therefore, not accounting for strength degradation should not be an issue.
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The mass used for the RUAMOKO wall model is the seismic mass of the building. This mass corresponds to the loading case $1.0E + 1.0D + 0.5L + 0.25S$, which is one of the two loading cases prescribed in the NBCC 2005 for seismic design (see Table 6.1 of the present document). The resulting total seismic weight of the building is 100468 kN, which is about 17% greater than that corresponding to seismic design loading case $1.0E + 1.0D$. The seismic mass of each storey is lumped at each floor level, as it is assumed that the floors of the building act as rigid diaphragms. The entire mass of the penthouse is lumped to the mass of the roof of the building. This results in twelve concentrated masses from floor levels 2 to 13 (building roof). These masses are given in Table B.4 as seismic weights.

The axial compression force acting on the wall is assumed constant and corresponds to the gravity loads of the loading case used to determine the seismic mass of the building. This force is used to determine the trilinear bending moment-curvature relationship for the wall member of each storey.

7.1.2 EFiCoS Cantilever Wall Model

As the RUAMOKO model, the EFiCoS cantilever wall model is a single-line stick model representing the whole cantilever wall structure. However, in this case, the wall is modelled with Bernoulli multilayered beam-line elements. These elements are located at the wall centroid.

Only one beam element discretization is used for the EFiCoS wall model. As shown in Figure B.3, this element discretization is the NSW3. The EFiCoS wall model is referred to as NSW3-E. Preliminary analyses have shown that the influence of the selected beam element discretization on predicted global structural responses is negligible. This would not be the case for structures with a softening behavior, as they are mesh dependent (Légeron et al., 2005). The beam element discretization of the EFiCoS wall model is selected to allow an inelastic flexural spreading along any possible plastic hinge locations.

As for the RUAMOKO model, the cantilever wall section of the EFiCoS model is composed only of the two C-shaped walls. Coupling beams are neglected. The resulting section is equivalent to an I-shaped section. As shown in Figure B.3, this section is divided
in a series of parallel concrete layers. A total number of 31 layers is used. The dimensions and reinforcing steel area of each concrete layer is given in Table B.5. These characteristics are assigned to each beam element composing the wall model.

The material constitutive models used for concrete and reinforcing steel are the Laborderie’s damage model (Laborderie, 1991) and a bilinear elastic-perfectly plastic model, respectively (see Fig. B.3). It is beyond the scope of this work to present the Laborderie’s model, but more information can be found in Laborderie (1991). Although no strain hardening is accounted for in the constitutive model for reinforcing steel, preliminary analyses have shown that this simple model is adequate for the present case. The values used in this work for the material parameters defining the previous constitutive models are given in Tables B.6 and B.7. These values are consistent with those used for the RUAMOKO cantilever wall model.

Material parameter values for concrete given in Table B.6 come from Légeron et al. (2005) and are for normal-strength concrete. Légeron et al. determined the values of the material parameters defining the behavior of concrete in compression by fitting the Laborderie’s model to monotonic concrete stress-strain curves measured from tests on over-reinforced beams made with normal-strength concrete. For the material parameters defining the behavior of concrete in tension, they determined the values of these parameters from empirical equations and by fitting the post-peak tensile response of a "numerical cylinder", using the Laborderie’s model, to the tension stiffening model proposed by Vecchio and Collins (1986).

Material parameter $Y_{01}$ of the Laborderie’s model controls the peak tensile strength of concrete and parameters $A_1$ and $B_1$ control the post-peak behavior in tension, that is, the tension stiffening of concrete. To simulate the LB and UB cases, values of the previous parameters are modified. For the LB case, where the concrete strength in tension is completely ignored, the $Y_{01}$ value is reduced to a value that is significantly lower than 145 Pa and does not produces numerical ill-conditioning, as EFICoS does not run when $Y_{01} = 0.0$. Moreover, $A_1 = 1.0 \text{ Pa}^{-1}$ is used, as suggested in Légeron et al. (2005), to inhibit the tension-stiffening effect. For the UB case, where the concrete tension-stiffening effect is accounted for, only the $A_1$ value is modified. The original $A_1$ value of $0.012 \text{ Pa}^{-1}$ reported in Légeron et al. (2005) for normal-strength concrete is changed for $0.002 \text{ Pa}^{-1}$. Figure B.4 shows the impact of this change on the lateral load-displacement
response of the wall model. From this figure, it is observed that the tension-stiffening effect increases significantly as $A_1$ is reduced from 0.012 to 0.002 Pa$^{-1}$. Moreover, the figure shows that the response corresponding to $A_1 = 0.012$ Pa$^{-1}$ is almost the same as that where tension stiffening of concrete is neglected ($A_1 = 1.0$ Pa$^{-1}$). This is in contradiction with experimental and numerical results reported in different works (Kwak and Kim, 2001; Adel and Ibrahim, 2002) performed on small- and large-scale RC cantilever wall specimens made of normal-strength concrete. In these works, pushover simulations not accounting for the tension-stiffening effect predict the onset of cracking at lateral load levels considerably less than those measured from previously uncracked test specimens. This means that $A_1 = 0.012$ Pa$^{-1}$ is not appropriate for the UB case. Also it appears that this value is not appropriate to simulate the tension stiffening of concrete in slender structural walls made of normal-strength concrete. Moreover, it seems that this value is not typical for normal-strength concrete, regardless of the type of structure. Actually La Borderie (1991) used $A_1 = 0.004$ Pa$^{-1}$ to simulate with EFICoS the flexural behavior of different reinforced concrete test specimens (beams, columns...) made with normal-strength concrete. In consequence, the $A_1$ value of 0.002 Pa$^{-1}$ used in the present project for the UB case was determined by fitting the lateral load-displacement responses predicted from EFICoS (see Fig. B.4) to those predicted from the RUAUMOKO wall model for the same case. This ensures the same base of comparison for the investigation. Curiously this value is the same as that reported in Légeron et al. (2005) for high-strength concrete.

The mass used for the EFICoS wall model is the same as that used for the RUAUMOKO wall model. The seismic mass of each storey is concentrated at each floor level of the building, as given in Table B.4. Moreover, the axial gravity force acting on the wall is the same as that used for the RUAUMOKO wall model. However, this time, this force is explicitly specified in the wall model.

### 7.1.3 Structural Response Assessment

Two methods of inelastic analysis are used to compute the structural responses of the RUAUMOKO and EFICoS wall models: a pushover analysis method and a time-history dynamic analysis method.
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Inelastic pushover analyses computed with EFiCoS are controlled by displacement while those computed with RUAUMOKO are controlled by force. This choice is for convenience matter only, as displacement or force control produces almost identical responses. Lateral load-displacement responses are computed for the nominal flexural resistance of the wall.

For dynamic analysis, the unconditionally stable Newmark constant acceleration step-by-step integration method is used together with the Rayleigh damping based on the initial elastic stiffness matrix. Only this damping model was available in the EFiCoS version used for this work. A 5% modal viscous damping ratio is assumed in the first and twelfth elastic mode of the cantilever wall. Preliminary RUAUMOKO analyses have shown that maximum global structural responses predicted with this damping model are relatively similar to those predicted with a damping model assuming a 2% modal viscous damping ratio in each elastic mode. Ground motion records used as input excitations are the NBCC 2005-compatible ground motion records for Montréal shown in Figures C.1 and C.2 in Appendix C of this document. Responses to ground motions are computed for the probable flexural resistance of the wall since the dynamic analysis conducted in Chapter 8 is based on this resistance, which is an idealization of the actual resistance in flexure of the wall.

Pushover Analysis Results

Lateral load-displacement responses predicted from the RUAUMOKO and EFiCoS wall models for the LB case are shown in Figure B.5. Figures B.5(a) and (b) present responses predicted from RUAUMOKO wall models NSW1-R, NSW2-R and NSW3-R meshed with one-component (Giberson) and general two-component beam elements, respectively. In each figure, these responses are compared to the response predicted from the EFiCoS wall model NSW3-E. In the same way, Figure B.6 shows the lateral load-displacement responses predicted from the previous wall models for the UB case. From Figures B.5 and B.6, the following observations can be made:

1. Except for some predictions, the match between RUAUMOKO and EFiCoS predictions is very good, independently of the beam element type and discretization used and the case, LB or UB, considered;
2. For the LB case (Fig. B.5(a)), finer is the one-component beam element discretization along the wall height softer is the flexural stiffness of the RUAUMOKO wall model. This mesh dependency is almost inhibited for the UB case (Fig. B.6(a)), that is, when the concrete tension-stiffening effect is accounted for;

3. Responses (Fig. B.6) predicted from models NSW2-R and NSW3-R for the UB case are approximately the same, independently of the type of beam element used;

4. For both LB and UB cases, RUAUMOKO models meshed with general two-component beam elements give good results, compared to EFiCoS predictions, especially when more than one element per storey is used. However, for the UB case (Fig. B.6(b)), these models overestimate slightly the flexural stiffness of the wall in the cracked-section portion of the lateral load-displacement response.

Tables B.8 and B.9 give base plastic hinge heights, at 2.5% lateral drift, predicted from the RUAUMOKO and EFiCoS wall models for the LB and UB cases, respectively. In RUAUMOKO models, flexural plasticity is lumped at beam element ends and represents the flexural yielding of the wall section. This allows an easy estimation of the hinging region, especially if the beam element discretization is relatively fine over this region. However, in EFiCoS, the flexural yielding of the wall section is not lumped into a simple global parameter. It is localized to each steel layer in the wall section. The estimation of the base plastic hinge height then is not as simple as for the RUAUMOKO models. EFiCoS results presented in Tables B.8 and B.9 give up to what storey from base yielding is predicted in tensile steel layers. Due to this difference, comparison between RUAUMOKO and EFiCoS results must be made with caution.

Tables B.8 and B.9 show that the EFiCoS model predicts yielding in tensile steel layers up to the base of storey 3 (S3) inclusive for the UB case and up to the base of S4 exclusive for the LB case. This lower prediction for the UB case is due to the contribution of concrete in tension. This contribution reduces stresses in tensile reinforcing bars and therefore their strain levels. The RUAUMOKO wall model cannot capture this difference since its global modelling approach. All RUAUMOKO predictions are lower than EFiCoS predictions, despite the fact that the EFiCoS model does not account for strain hardening in reinforcing steel. Nevertheless, RUAUMOKO predictions for the UB case are similar to the respective EFiCoS prediction, independently of the beam element type and discretization used. For
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the UB case, all RUAUMOKO models predict flexural yielding up to the base of S2 inclusive. The same prediction is obtained for the LB case with all RUAUMOKO models meshed with general two-component beam elements. However, for this case, the predicted base plastic hinge height decreases with increasing the number of one-component beam elements in the RUAUMOKO wall model.

All previous pushover results show that, in comparison with the EFICoS predictions:

1. the RUAUMOKO wall model meshed with general two-component beam elements can give very good pushover predictions, independently of the beam element discretization used and the case, LB or UB, considered. Pushover predictions get better when more than a single element per storey is used, especially over the expected hinging region; and

2. finer is the one-component beam element discretization along the wall height softer is the flexural stiffness of the RUAUMOKO wall model, except if the concrete tension-stiffening effect is accounted for. For the latter case, very good pushover predictions can be obtained, especially if a relatively fine element discretization is used over the expected hinging region. The one-component beam element discretization should be limited to a single element per storey when the concrete strength in tension is neglected.

Dynamic Analysis Results

RUAUMOKO and EFICoS wall models used for pushover analysis are used also for dynamic analysis. However, for dynamic analysis, only the beam element discretization NSW3 is considered for the RUAUMOKO wall model. This element discretization is the most appropriate one to represent the formation of plastic hinges at any possible location along the wall height.

Table B.10 gives the vibration periods of the first three elastic lateral modes of the RUAUMOKO and EFICoS wall models. It shows a very good match between the lateral mode period values of both models, though period values of the EFICOS model are slightly less. This means that the EFICoS model is a bit stiffer in flexure than the RUAUMOKO model.
Table B.11 gives dynamic analysis results obtained from the EFiCoS model (NSW3-E) for the LB and UB cases. Results presented in this table are the maximum top wall displacement, $\Delta_{\text{max}}$, the maximum wall base shear force, $V_{h,\text{max}}$, and the location of the hinging region(s). From this table, it is observed that none of the input ground excitations produced the formation of plastic hinges in the EFiCoS wall model. The $\Delta_{\text{max}}$ predictions for the LB and UB cases are in general almost the same. Actually the mean $\Delta_{\text{max}}$ value for both cases is about 75 mm. However, higher $V_{h,\text{max}}$ values are predicted for the UB case. One of these values is as high as 18810 kN. The mean $V_{h,\text{max}}$ values for the LB and UB cases are about 11370 and 16000 kN, respectively. It is noted that the latter value is about 27% greater than the design (probable) base shear for the cantilever wall, which is 12600 kN (see Table 6.6). Moreover, one third of the $V_{h,\text{max}}$ predictions for the LB case are greater than 12600 kN.

Tables B.12 and B.13 give dynamic analysis results obtained from the RUAUMOKO wall model NSW3-R meshed with one-component and general two-component beam elements, respectively. As for the EFiCoS model, the RUAUMOKO model predicts no plastic hinges in the wall under the input ground excitations, independently of the case, LB or UB, considered and the type of beam element used. Table B.12 shows that the $\Delta_{\text{max}}$ predictions, for the LB and UB cases, obtained from the RUAUMOKO model meshed with one-component beam elements are, on average, about 2.8 and 2.0 times greater than those obtained from the EFiCoS model, respectively. This significant difference was expected for the LB case but not for the UB case, based on the pushover results. When the RUAUMOKO model is meshed with general two-component beam elements, these ratios drop to about 2.0 and 1.5, respectively. The $V_{h,\text{max}}$ predictions, for the LB case, obtained from the RUAUMOKO models meshed with one-component and general two-component beam elements are, on average, about 32% and 24% lower than those from the EFiCoS model, respectively. However, for the UB case, these percentages drop to 15% and 10%, respectively. Although the $V_{h,\text{max}}$ predictions, for the UB case, obtained from RUAUMOKO are lower than those obtained from EFiCoS, they are, on average, greater than about 10% of the design (probable) base shear for the cantilever wall.

Previous comparisons based on some maximum dynamic predictions show that the best match between RUAUMOKO and EFiCoS results is obtained when the RUAUMOKO model is meshed with general two-component beam elements and accounts for the tension-
stiffening effect. The same statement can be made with regard to response waveform. Figure B.7 shows the top wall displacement responses, predicted from RUAUMOKO and EFiCoS models for the UB case, to MTL2-M6, MTL1-M7 and MTL2-M7 ground excitations. From this figure, a better match between RUAUMOKO and EFiCoS predictions is observed when the RUAUMOKO model is meshed with general two-component beam elements. Although peak displacements are sometimes considerably different, response waveforms are relatively similar. This match is not as good for the base shear force response. This could be expected as shear force is generally more sensitive to higher modes response and varies more rapidly with time relative to the accompanying displacement. Moreover, several parameters, such as damping and shear modelling, can highly influence shear force response. This high sensitivity may explain why the $V_{h,max}$ predictions obtained from RUAUMOKO and EFiCoS for the UB case are, on average, greater than about 10% and 27%, respectively, of the design (probable) base shear for the cantilever wall.

As noted previously, none of the NBCC 2005-compatible ground motion records for Montréal used as input excitations for the dynamic analysis produced the formation of plastic hinges in the RUAUMOKO and EFiCoS wall models. Is this due to the damping model used for analysis or the relatively great overstrength in flexure of the cantilever wall? This question is discussed in Chapter 8. But what are the predictions when plastic hinges form? When it happens, is the tension-stiffening effect still an important parameter to account for in the wall models? In order to answer to these questions, the North-South component of the well-known 1940 El Centro earthquake record (ELCENTRO) is used as input motion. As shown in Figure C.3, this earthquake is much more severe than the NBCC 2005 acceleration spectrum for Montréal used for seismic design. Such severe earthquake should produce plastic hinges in the wall models.

Table B.14 gives dynamic analysis results obtained from RUAUMOKO and EFiCoS wall models when subjected to ELCENTRO ground excitation. These results show that plastic hinges did form in all models, except in the RUAUMOKO model meshed with one-component beam elements and free of any concrete strength in tension. Unlike RUAUMOKO models, the EFiCoS model predicts plastic hinges at the same locations for the LB and UB cases. Moreover, predicted plastic hinges are not constrained only at the base of the wall. A plastic hinge located in the vicinity of floor level 8 is also predicted.
Similar predictions were obtained by Tremblay et al. (2001) and Renaud (2004) when their 12-storey ductile RC cantilever wall models were subjected to El Centro earthquake. RU-AUMOKO predicts plastic hinges at the same locations as those predicted from EFICoS only when the tension-stiffening effect is accounted for in the wall models. Each wall model predicts almost the same $\Delta_{\text{max}}$ values (see Table B.14) and top wall displacement response waveforms (see Fig. B.8) for the LB and UB cases. This means that the tension-stiffening effect has no significant influence on the top wall displacement response predicted from each wall model. Nevertheless, overall the best match between RU-AUMOKO and EFICoS predictions are obtained when the RU-AUMOKO wall model is meshed with general two-component beam elements and accounts for the tension-stiffening effect.

From Table B.14, it is interesting to point out that the $V_{b,\text{max}}$ predictions obtained from the RU-AUMOKO and EFICoS wall models for the LB case are significantly greater than those for the UB case. This contrasts with results presented in Tables B.11, B.12 and B.13. Although for the EFICoS model the tension stiffening of concrete has no significant effect on top wall displacement and plastic hinge predictions, it plays a major role in $V_{b,\text{max}}$ predictions. Table B.14 shows a drop from 24670 to 16540 kN of the predicted $V_{b,\text{max}}$ value when the tension-stiffening effect is accounted for in the EFICoS model. It is interesting to point out that the latter value is quite similar to $V_{b,\text{max}}$ values presented in Table B.11 for the UB case, even if ELCENTRO is a much more severe earthquake than the selected NBCC 2005-compatible earthquakes for Montréal. However, for the LB case, there is no such similitude. Actually the $V_{b,\text{max}}$ value of 24670 kN in Table B.14 is about twice those given in Table B.11 for the LB case. All these observations for the EFICoS $V_{b,\text{max}}$ predictions also apply to RU-AUMOKO predictions (Table B.14 versus Tables B.12 and B.13). The opening and reclosure of cracks simulated in the EFICoS model may be in part the explanation of the significant increase in the EFICoS $V_{b,\text{max}}$ predictions for the case where the concrete strength in tension is ignored (LB case). However, similar increases are predicted from the RU-AUMOKO models even if this cracking simulation is not included in the models. The assumption of linear shear in the wall may be another explanation. This assumption is reasonable until flexural yielding occurs. Actually laboratory tests (Oesterle et al., 1977) of isolated RC walls have indicated that flexural yielding is usually accompanied by shear yielding. Higher mode effects certainly play a major role in increased $V_{b,\text{max}}$ predictions since the the base shear force response is very sensitive to these effects, particularly when base yielding occurs under a very high seismic
intensity (Seneviratna and Krawinkler, 1994). Despite the causes of these increases, it appears that, when it is accounted for, the tension-stiffening effect ensures numerically more stable dynamic base shear force predictions and likely more reliable results. Not accounting for this effect produces a significant increase in dynamic base shear force predictions at a seismic intensity considerably greater than that at design level.

**Important Findings from the Modelling Assessment**

Results presented in Section 7.1.3 lead to the following important findings with regard to inelastic modelling:

1. When general two-component beam elements are used to mesh the RUAMOKO wall model, any of the beam element discretizations (see Fig. B.2) selected for the modelling assessment can be used, though a relatively fine element discretization in the expected hinging region is more suitable for inelastic seismic analysis. However, the element discretization should be limited to a single element per storey (NSW1) when one-component beam elements are used to represent the wall, unless the tension-stiffening effect is accounted for in the wall model. In this case, an element discretization similar to NSW2 or NSW3 can be used;

2. In comparison with the EFiCoS predictions, the best RUAMOKO predictions for both inelastic pushover and dynamic analysis were obtained when the wall model was meshed with general two-component beam elements, according to a relatively fine element discretization, and accounted for the tension-stiffening effect;

3. When it is accounted for, the tension-stiffening effect ensures more numerically stable dynamic base shear force predictions for the studied cantilever wall model, even if the input earthquake excitation is much more severe than that used for design and produces the formation of plastic hinges. Otherwise, a significant increase in dynamic base shear force predictions occur with increasing seismic intensity above design level.

Although previous findings are based on simulations of a single RC cantilever wall structure, they suggest that the tension-stiffening effect be taken into account when sim-
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ulating the inelastic seismic behavior of RC shear wall structures. More rigorous investigations should be performed on this issue.

Results presented in Section 7.1.3 also highlight important findings with regard to seismic predictions for the studied cantilever wall:

1. No plastic hinges may form in the wall under the selected NBCC 2005-compatible earthquakes for Montréal, though the structure is designed and detailed to be ductile under such earthquakes;

2. The inelastic mechanism may not be constrained at the base of the wall, as assumed for design, when the latter is subjected to earthquakes considerably more severe than the design level;

3. Base shear forces significantly greater than the design base shear (12600 kN) may be expected under the selected NBCC 2005-compatible earthquakes for Montréal.

7.2 Structural Models for Inelastic Seismic Analysis

This section presents the structural models used in RUAUMOKO to verify the seismic design and to assess the seismic performance of the core wall through inelastic static (pushover) and time-history dynamic analyses. Results of these analyses are presented in Chapter 8 of this document. The inelastic modelling adopted for the structural models is based on the important findings obtained from the modelling assessment conducted in Section 7.1. Unless otherwise specified in the text, the structural models are developed for a factored, nominal and probable flexural resistance, as defined in the CSA A23.3-04.

7.2.1 Cantilever Wall Model

The cantilever wall model is the same wall model as the one used for the modelling assessment. A detailed description of this model is given in Section 7.1.1. However, the
following modelling characteristics are used here:

- The wall model is meshed with general two-components beam elements according to the element discretization NSW3 (see Fig. B.2); and

- The wall model accounts for the tension-stiffening effect. This is performed by using the trilinear bending moment-curvature relationship proposed by Adebar and Ibrahim (2002) to take into account this effect.

It is noted that the shear deformation of the wall is assumed to be linearly elastic. This assumption is reasonable since the very low ductility demand on the wall expected at design level (see Table 6.7). As indicated in Table B.1, the elastic shear stiffness of the wall is based on the effective shear area of the wall section.

The resulting cantilever wall model is shown in Figure B.9 and is referred to as NSW-R. The elastic dynamic characteristics of this model are given in Table B.15.

### 7.2.2 Building Model in the Cantilever Wall Direction

As indicated at the beginning of this chapter, a 2D model of the whole building structure in the N-S direction is considered for the seismic analysis in order to study the response of the cantilever wall when the stiffness of all structural components of the building is accounted for in the modelling. It is noted that this model is used only for the inelastic time-history dynamic analysis. Consequently, the model is developed for a probable flexural resistance only.

The 2D model of the building, referred to as NSB-R, is shown in Figure B.10. As the building structure is symmetric about its center, the model is composed of only three frames, one exterior frame and two interior frames. Each frame represents two identical frames oriented along the N-S column lines of the building. The lateral displacements of frames are kinematically constrained at each floor level to be the same. This is based on the assumption that the floors of the building act as rigid diaphragms.
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The model of the cantilever wall structure is the wall model NSW-R presented in Section 7.2.1. All other structural components of the building are modelled with one-component beam elements, as shown in Figure B.10. Elastic material properties used for these components are the same as those used for the cantilever wall structure (see Table B.1). Cross-sectional dimensions used for the columns are given in Table 5.1 while those used for the slabs and spandrel beams are given in Figure B.10. For the interior frames, the effective slab width for each column line is taken as 1.0 m, except for the slab surrounding the core wall. For this area, the effective slab width for each column line is taken as the column strip, which is estimated to be 3.0 m. Beam elements representing the effective slabs in this area have rigid extensions at one end to account for the finite width of the wall structure. The effective flange width of spandrel beams is estimated as recommended in Paulay and Priestley (1992) for seismic analysis. The moment of inertia of the beam elements representing the slabs, spandrel beams and columns is reduced, as specified in the CSA A23.3-04, to account for cracking. The effective (cracked) moment of inertia of these elements is as follows: 0.20 \( I_g \) for slabs, 0.40 \( I_g \) for spandrel beams and 0.70, 0.65 and 0.60 \( I_g \) for columns of storeys 1 to 3, 4 to 9 and 10 to 12, respectively.

The hysteretic behavior in flexure of the beam elements representing the columns and the spandrel beams is idealized with a simple elasto-perfectly plastic hysteresis model. The probable flexural (yield) strengths of each member are used as the yield bending moments of the primary curve of this hysteresis model. These flexural strengths are determined as specified in the CSA A23.3-04 and using MNPHi. The deformations of the beam elements representing the slabs are idealized as linearly elastic. The linear shear assumption is used for all elements.

The mass used for the building model is the same as that used for the cantilever wall model NSW-R. The seismic mass of each storey is concentrated at each floor level of the building, as given in Table B.4. The resulting elastic dynamic characteristics of the NSB-R model are given in Table B.15. This table indicates that the fundamental period of this model is only about 5% lower than that of the cantilever wall model. This shows that the added stiffness from the structural elements not part of the wall system has little influence on the elastic dynamic characteristics of the building structure.
7.2.3 Coupled Wall Model

The coupled wall system is composed of two C-shaped walls connected by coupling beams. The following presents the structural modelling of each member of this system.

C-Shaped Walls

The modelling of each C-shaped wall is almost the same as that of the cantilever wall presented in Section 7.2.1. Actually each C-shaped wall is modelled as follows:

- The hysteretic behavior in flexure of the wall member of each storey is idealized through the trilinear hysteretic model SINA. No strength degradation is accounted for as little inelastic deformation in flexure of the C-shaped walls is expected. See Section 7.1.1. The shear deformation of wall members is assumed to be linearly elastic. This assumption is reasonable since the very low ductility demand on the wall system expected at design level (see Table 6.7). As indicated in Table B.1, the elastic shear stiffness of a C-shaped wall is based on the effective shear area of the wall section;

- The primary curve of the SINA hysteretic model is defined by the trilinear bending moment-curvature relationship proposed by Adebar and Ibrahim (2002) and accounts for the tension-stiffening effect (see Fig. 4.5). The trilinear relationship is modified to account for strain hardening in reinforcing steel. Values of the parameters defining this relationship are determined with the same approach as that used for the cantilever wall model. Two trilinear relationships are defined for storey 7 due to the change of concentrated vertical reinforcement at this storey. See Section 7.1.1;

- Wall members are modelled with general two-component beam elements. The beam elements are located at the member centroids. The elastic material and section properties used for these elements are given in Table B.1;

- The beam element discretization NSW2 (see Fig. B.2) is used since this discretization has shown in a previous work (Renaud, 2004) to be appropriate for a similar ductile coupled wall system.
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The main difference between the modelling of the cantilever wall and that of the C-shaped walls of the coupled wall system comes from the axial force induced in the walls through the coupling action. For a heavily coupled wall system, such as the one in this project, this force is quite large. As a result, interaction of axial and flexural behavior becomes an important factor that must be accounted for in the modelling. However, in RUAUMOKO, the only two beam-column elements that can take into account this interaction for reinforced concrete members are limited to one-component elements. Moreover, preliminary analyses with the only beam-column element adequate for the unsymmetrical section of the C-shaped walls predicted strange responses likely due to ill-conditioning problems of the element.

Consequently, a modelling approach was developed to explicitly account for the axial-flexural interaction with general two-component beam elements. Under a severe earthquake ground motion, the instantaneous net axial forces in the walls of a coupled wall system will rarely be equal to the axial compression forces due to gravity loads, though they will oscillate around the latter. For a heavily coupled wall system, this variation is generally quite large. So large that the instantaneous net axial force may be in tension rather than in compression for a very short duration. As a result, for such system, the minimum and maximum net axial forces in a wall are two important limit cases. These two cases are used in this project to determine the trilinear bending moment-curvature relationships for the C-Shaped wall members.

The minimum and maximum net axial forces in a wall can be estimated by adding or subtracting to the axial forces due to gravity loads the maximum possible earthquake-induced axial force. The latter occurs when all coupling beams above the wall section of interest develop their maximum flexural strength. In this project, this strength is taken as the factored, nominal or probable flexural yield strength of the coupling beam, depending on the type of resistance considered for analysis. For a given type of resistance, the maximum possible earthquake-induced axial force is taken as the sum of the end shears corresponding to the flexural strength in the coupling beams above the wall section of interest. For the probable resistance, this sum is reduced using Equation (3.9) since it is unlikely that, for a 12-storey coupled wall building, all coupling beams above the level considered will develop their probable flexural strength at the same instant in time. It is noted that the the gravity loads used to calculate the minimum and maximum net axial
forces in the C-shaped walls correspond to the same loading case as that used to determine the seismic mass of the building.

**Coupling Beams**

Coupling beams are modelled with one-component beam elements. The one-component beam element is adequate for this member. In fact, the formulation of this element is based on a deformed shape in flexure that is similar to that experienced by a coupling beam under an earthquake loading, that is, a double curvature. The two coupling beams at each floor level are represented by a single one-component beam element. The element is located at the member centroids. The end regions of each element are represented with infinitely rigid extensions in order to account for the finite widths of the adjoining C-shaped walls.

The elastic material and section properties used for each beam element are given in Table B.1. The shear areas \( A_s \) and moments of inertia \( I \) given in this table for coupling beams are reduced section properties that account for cracking. These properties were obtained by scaling down the gross section properties of the coupling beams with reduction factors. The reduction factors used for the moment of inertia were determined from the bending moment-curvature responses of the coupling beams computed with MNPHi. The reduction factors used for the shear area were determined from a simple rule of three between the previous reductions factors and those specified in the CSA A23.3-04 for a diagonally reinforced coupling beam. For such beam, the CSA A23.3-04 specifies reduction factors of 0.45 and 0.25 for the shear area and moment of inertia, respectively (see Table 3.4 of the present document). The reduction factors for the shear area were obtained by scaling those for the moment of inertia with the ratio 0.45/0.25 \( (=1.8) \).

The modelling of the inelastic deformations of coupling beams is limited to flexure. Their shear deformation is assumed to be linearly elastic. Their axial rigidity is assumed to be infinite since the lateral displacements of the adjoining walls are kinematically constrained at each floor level to be the same. This is based on the assumption that the floors of the building act as rigid diaphragms.

The bilinear modified Takeda model is used to simulate the hysteretic response in flex-
ure of coupling beams. The primary curve of this model is taken as an elastic-perfectly plastic response. The yield bending moment of this response is taken as the resisting moment at the supports of the two coupling beams at the floor level considered. This resisting moment is calculated at the development of the yield strength in the diagonal bars assuming that these bars resist all the forces at beam ends. As shown in Figure 4.4, the modified Takeda model accounts for stiffness degradation but not for strength decay. Strength degradation of coupling beams is ignored in this project as little inelastic rotational demand on the coupling beams is expected (see Table 6.7). In the modified Takeda model, the inelastic stiffness during unloading and reloading is controlled with the parameters $\alpha$ and $\beta_r$, respectively. These parameters are difficult to determine without test results. However, various studies (Saatcioglu et al., 1983; Chaallal and Gauthier, 2000) showed that variations in these parameters, within the range observed in tests, do not significantly affect dynamic response. The $\alpha$ and $\beta_r$ values used for this project are 0.1 and 0.5, respectively. These values are those used by Harries et al. (1998) for concrete coupling beams diagonally reinforced in accordance with the seismic provisions of the CSA A23.3-94 and having a span-to-depth ratio (1.7) very similar to that (1.8) of the coupling beams of this project, though the flexural strengths of the latter are less than that of the former. According to Harries et al., these $\alpha$ and $\beta_r$ values were obtained by comparing predicted responses with test results.

As indicated previously in Section 7.1.1, a plastic hinge length can be specified in RUAUMOKO for each plastic rotational end spring of a one-component beam element. Since the primary response in flexure of a coupling beam is idealized as an elastic-perfectly plastic response, the relationship in RUAUMOKO between the inelastic rotation, $\theta_i$, and the inelastic curvature, $\phi_i$, of a plastic rotational end spring is:

$$\theta_i = H \cdot \phi_i$$ (7.1)

where $H$ is the specified plastic hinge length.

Using the simple truss model shown in Figure 7.1, Paulay (2001) developed relationships to estimate the yield chord rotation, $\theta_{by}$, at the supports of a diagonally reinforced coupling beam. These relationships assume that the diagonal members resist all forces at beam ends, the yield displacement of the beam can be based solely on the elongation of
the diagonal members and the deformations due to diagonal compression can be neglected as they are generally very small. From these relationships, the plastic hinge length, $H$, in Equation (7.1) can be estimated as follows:

$$H = \frac{(S/\cos \alpha) + 20 d_b}{\sin 2\alpha} \left( \frac{D}{S} \right)$$

(7.2)

where $D$ is the vertical distance between both diagonal members at beam end, $S$ is the clear span of the beam, $\alpha$ is the inclination of the diagonal members with respect to horizontal, and $d_b$ is the diameter of the diagonal bars. The term $20 d_b$ in Equation (7.2) is to account for anchorage deformations originating in the adjacent walls. This equation was used to calculate the plastic hinge lengths for the coupling beams of the studied coupled wall. The resulting plastic hinge lengths are 1.4 and 1.3 m for the coupling beams located at floor levels 2 to 10 and 11 to 14 (penthouse), respectively. These lengths are specified in the coupled wall model.

![Truss model](image)

Figure 7.1: Truss model used by Paulay (2001) for a diagonally reinforced coupling beam

**Coupled Wall System**

Figure B.9 shows the coupled wall model resulting from the structural modelling of the members of the studied wall system. This model is referred to as EWW-R.

The mass used for this wall model is the same as that used for the cantilever wall model NSW-R. The seismic mass of each storey is concentrated at each floor level of the building, as given in Table B.4. The resulting elastic dynamic characteristics of the EWW-R model are given in Table B.15.
Chapter 8

Inelastic Seismic Analysis

This chapter presents the inelastic pushover and time-history dynamic analysis results obtained from the structural models presented in Section 7.2 of this document, the discussion on these results and the important findings resulting from this discussion. The computer program RUAUMOKO was used to perform these analyses. A detailed description of the input control parameters (time step, damping...) and earthquake loadings used for the numerical analyses is given in Section 5.3.5. Inelastic analysis responses are presented in Appendix D.

8.1 Pushover Analysis Results

8.1.1 Cantilever Wall

Figure D.1(a) illustrates the base shear force versus lateral drift at building roof predicted from the cantilever wall model NSW-R for the factored, nominal and probable flexural
resistances. The NSW-R model was laterally pushed over up to a roof drift of 2.5%, which is the overall drift limit permitted by the NBCC 2005 for the studied building.

Figure D.1(a) shows, as expected, that the behavior in flexure of the cantilever wall structure is elastic when the applied base shear force is equal to the NBCC 2005 seismic design base shear of 2062 kN for this loading direction. Actually the structure is uncracked in flexure at this base shear force level. From the probable response at drift limit, an overstrength factor can be estimated as $5000/2062 \approx 2.4$. This value is greater than the $R_o$ value of 1.6 used for design. This difference is mainly due to the excess strength in flexure arising from the required minimum reinforcement in the assumed base plastic region of the wall. This contribution is not accounted for in the formulation of $R_o$ (see Section 3.1.4).

At design lateral drift, which is only 0.2%, the concrete wall structure is cracked but has not yielded in flexure, for any resistance. At this drift, Figure D.2(a) shows that cracking in flexure has occurred up to the base of storey 3 inclusive. However, for a nominal resistance, the wall section at the base of the wall has almost yielded. This is in line with the very low inelastic rotational demand (0.001 rad.) predicted for the seismic design of the cantilever wall (see Table 6.7). The cantilever wall structure then has still a lot of residual strength at design drift, as shown in Figure D.1(a). Based on Tables 2.1 and 2.2, the overall damage state of the cantilever wall structure at design drift corresponds to the performance level "operational" rather than "near collapse", which is the performance level expected for the seismic event (2500-year return period) used for design. At the overall drift limit of 2.5%, Figure D.2(b) shows that flexural yielding occurs at the base of the wall and up to the base of storey 2 inclusive, according to the sequence indicated in this figure. In addition, the wall is cracked in flexure up to the base of storey 5 inclusive. The flexural yielding region lies within the assumed base plastic hinge region, which is over the three first storeys from the base of the wall. Table D.1 gives the maximum curvature ductility ratios predicted in this region. These ratios are equal to or slightly greater than the ultimate curvature ductility ratios for the base of the wall. This means that the flexural capacity of the cantilever wall is reached at an overall drift of 2.5%. The overall damage state of the cantilever wall structure at this drift then corresponds remarkably well to the performance level "near collapse" described in Tables 2.1 and 2.2.
Overall the NSW-R model gives predictions that are in line with the expectations, that is, significant overstrength in flexure due to the required minimum reinforcement, onset of yielding at design drift for a nominal resistance and first yielding at the base of the wall. This confirms the validity of this wall model to adequately simulate the overall flexural behavior of the ductile cantilever wall structure. Predictions also show that this structure has still a lot of residual strength at design drift. This performance level is considerably better than that expected for the 2500-year return period event used for seismic design. In addition, no yielding is expected at design drift for a probable resistance.

8.1.2 Coupled Wall

Figure D.1(b) illustrates the base shear force versus lateral drift at building roof predicted from the coupled wall model EWW-R for the factored, nominal and probable flexural resistances. The EWW-R model was laterally pushed over up to a roof drift of 1.13%. This drift limit is less than half of the 2.5% drift limit permitted by the NBCC 2005 for the studied building. The reason is that the overall drift of the coupled wall is limited by the inelastic rotational capacity of coupling beams. The CSA A23.3-04 limits this capacity to 0.04 rad. for diagonally reinforced coupling beams. The overall drift limit of 1.13% corresponds to this inelastic rotational capacity.

Figure D.1(b) shows, as expected, that the behavior in flexure of the coupled wall structure is elastic when the applied base shear force is equal to the NBCC 2005 seismic design base shear of 1760 kN for this loading direction. Actually the structure is uncracked in flexure at this base shear force level. From the probable response at drift limit, an overstrength factor can be estimated as \(4360/1760 \approx 2.5\). This value is greater than the \(R_o\) value of 1.7 used for design. Once again, this difference is largely due to the great excess strength in flexure arising from the required minimum reinforcement in the assumed base plastic region of the wall.

Figure D.1(b) shows that the design and the detailing of the coupled wall structure are adequate to produce the desired inelastic mechanism, that is, yielding of all coupling beams prior to walls. Moreover, the tension wall yields prior to the compression wall, as expected. The sequences of hinge formation at design drift and drift limit are indicated
in Figure D.2(a) and (b), respectively. As shown in this figure, all beams have yielded at design drift but none of the walls have. However, both walls are cracked in flexure at their base. Table D.2 indicates that the predicted maximum curvature ductility ratios in beam hinges at design drift are lower than 3 for any considered resistance and significantly less than the ultimate curvature ductility ratio of 10. The predicted ratios correspond to inelastic rotational demands ranging from about 0.005 and 0.008 rad. These low demands were expected and match remarkably well the inelastic rotational demand of 0.007 estimated for seismic design (see Table 6.7). The coupled wall structure then has still a large residual strength at design drift, as shown in Figure D.1(b). Based on Tables 2.1 and 2.2, the overall damage state of the coupled wall structure at this drift corresponds to the performance level "operational" rather than "near collapse", which is the performance level expected for the seismic event (2500-year return period) used for design. At drift limit, Figure D.2(b) shows that both walls have finally yielded at their base. Table D.3 shows that the predicted maximum curvature ductility ratios in beam hinges at drift limit equal to or exceed 10. Those in wall hinges do not exceed the ultimate curvature ductility ratios for the walls. The overall damage state of the coupled wall structure at drift limit (1.13%) corresponds relatively well to the performance level "life safe" described in Tables 2.1 and 2.2.

Figures D.3 and D.4 give the redistribution of base moment and base shear between walls predicted for a probable resistance, respectively. As expected, Figure D.3 shows that the main moment resisting mechanism in the coupled wall system is obtained from the coupling action. The base moment produced by this action is estimated to be about 67% and 57% of the total base overturning moment at design drift and drift limit, respectively. According to the NBCC 2005, a ductile coupled wall system is considered fully coupled when at least 66% of the total base overturning moment is carried by the coupling action. This means that the studied wall system can be considered as fully coupled up to design drift, as estimated for design. Actually it was estimated when designing that almost 80% of the total base overturning moment was carried by the coupling action (see Table 6.4). From Figure D.3, it appears that this 80% overestimates the base coupling moment predicted at any lateral drift. It is of interest to note from this figure that the wall system becomes partially coupled, as defined by the NBCC 2005, for lateral drifts greater than 0.18%. In fact, beyond this drift, it is estimated that the coupling action carries less than 66% of the total base overturning moment. Moreover, at design drift, the base moment
resisted by the compression wall represents about 20% of the total base overturning moment and is about 30% greater than that resisted by the tension wall. From Figure D.4, the compression wall is estimated to be resisting a maximum of about 65% of the total base shear force before reaching design drift and almost 70% at drift limit. These significant base moment and shear redistributions between walls are typical for ductile heavily coupled wall systems (Shiu et al., 1984; Paulay and Priestley, 1992).

Overall the EWW-R model gives predictions that are in line with the expectations, that is, significant overstrength in flexure due to the required minimum reinforcement, yielding of all coupling beams prior to walls, low inelastic rotational demands in coupling beams at design drift, yielding of the tension wall prior to the compression wall when the wall system is pushed beyond design drift, a moment resisting mechanism mainly controlled by the coupling action and significant base moment and shear redistributions from the tension wall to the compression wall. This confirms the validity of this wall model to adequately simulate the overall flexural behavior of the ductile coupled wall structure. Predictions also show that this structure has still a large residual strength at design drift. This performance level is considerably better than that expected for the 2500-year return period event used for seismic design.

### 8.2 Time-History Dynamic Analysis Results

Time-history dynamic analysis results are presented in terms of mean and maximum peak values of the structural response predictions. In order to assess the variability in predictions, the mean peak responses (M) plus or minus one standard deviation (SD) and the associated coefficient of variations (COVs) are also given (Note: COV = SD/M). The M+SD values represent an 84% level of confidence that peak predictions are less than or equal to these values. Variability in peak predictions are due to the different profiles of ground motion records used for analysis, even if these records produce very similar spectral accelerations, as shown in Figures C.1 and C.2. For a given high ground motion acceleration intensity, the record-to-record variability is generally the most dominant variability in seismic demand predictions, but the variability due to modelling uncertainty (structural properties...) can also be large enough to be important, especially at low in-
tensity level (Lee and Mosalam, 2005). The latter variability is however not accounted for in this project.

In theory, maximum predicted peak response values should match or be lower than the design values determined from the provisions of the NBCC 2005 and the CSA A23.3-04 for seismic design. Actually these provisions are intended to ensure conservative designs. Nevertheless, maximum predicted peak values may exceed code design values. Insignificant excesses are not an issue since all the uncertainties (ground motions, structural properties...) that are inherent to inelastic analysis. Considerable excesses need for sure a special attention but should be addressed carefully as they may be due to a single input motion. However, if mean predicted peak values also significantly exceed code design values, then this may highlight a serious issue with the seismic code provisions.

8.2.1 Cantilever Wall

The predictions presented in this section were obtained from the cantilever wall model NSW-R.

Overall Drift and Plastic Hinges

Table D.4 gives the maximum roof drift ($\Delta_{max}$), plastic hinges and associated maximum curvature ductility ratios ($\mu_{p}^{max}$) predicted for 1% and 2% damping. This table shows that in general roof drift predictions are reasonably similar to the design lateral drift of 0.20%, though some predictions are about two times greater than design drift, for both damping levels. These large differences are meaningless considering the very low drift values and the significant dispersion of these values (COV $\approx$ 0.30). For 2% damping, no plastic hinges are predicted in the wall structure, even if a maximum roof drift of 0.41% is predicted from the MTL2-M7 record. Actually, over a roof drift of 0.30%, Figure D.1(a) shows that a plastic hinge should form at the base of the wall if the wall response is mainly governed by its first lateral mode of vibration. This indicates, as expected, that higher modes play a major role on the cantilever wall response and that the relation overall drift-hinging is not appropriate for this wall structure.
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For 1% damping, Table D.4 shows that none of the ground motion records for M6 earthquakes at R=30 km (short-period events) produced the formation of plastic hinges in the wall model. However, two of the three ground motion records for M7 earthquakes at R=70 km (long-period events) did. This could be expected since the fundamental period of vibration of about 1.6 s of the wall model lies within the period range covered by these records. Predictions indicate that the formation of plastic hinges in the wall structure is not constrained at the base of the wall, as assumed for design. Under the MTL1-M7 ground motion, an additional plastic hinge is predicted at the base of storey 8. It is of interest to note that the $\mu_{\phi}^{\text{max}}$ of 7.0 predicted in this hinge is significantly greater than the $\mu_{\phi}^{\text{max}}$ of 1.5 predicted in the hinge at the base of the wall. This means that the curvature ductility demand in the hinge at storey 8 is greater than that in the hinge at wall base. The cantilever wall structure is not designed for this inelastic mechanism. Plastic hinge predictions also indicate that the curvature ductility demand at design level to be expected at the base of the wall is very low ($\mu_{\phi}^{\text{max}} \leq 1.5$).

Interstorey Drift

Figure D.5 gives maximum and M±SD predicted peak interstorey drift (PID) responses to all input motions for 1% and 2% damping. Predictions are compared to the design interstorey drifts, which were determined from the linear modal response spectrum method specified in the NBCC 2005, using the SRSS combination. This figure shows that maximum PID predictions for 1% and 2% damping are almost the same, except above storey 7. Above this storey, maximum PID predictions for 1% damping are larger than those for 2% damping, particularly at storey 8. This is due to the relatively great curvature ductility demand in the hinge predicted at the base of storey 8. Figure D.5 also shows that the design interstorey drifts considerably underestimate the maximum PID predictions for 1% and 2% damping, especially the PID of almost 1% predicted at storey 8 for 1% damping. The design interstorey drifts cannot predict this relatively large PID since they are determined from a linear analysis method. Although the maximum predicted PID values are underestimated by the design values, they are considerably less than the 2.5% interstorey drift limit defined by the NBCC 2005. It is of interest to note that the design interstorey drifts also underestimate the mean PID predictions for 1% and 2% damping, though this underestimation is considerably less than that for the maximum PID predictions. Nevertheless, design interstorey drifts are either equal to or larger than the M-SD
PID predictions. For 2% damping, the mean PID predictions are relatively similar to the design interstorey drifts.

Figure D.5 shows large variabilities in PID predictions, especially at upper storeys. An important difference between maximum and M+SD PID predictions is observed. It is of interest to note that maximum PID predictions are due to a single ground motion record, the MTL2-M7 record. Figure D.6 shows that the maximum PIDs predicted from this record are considerably larger than the mean PIDs predicted from all MTL-M7 records (M7 earthquakes at R=70 km) and than the maximum PIDs predicted from all MTL-M6 records (M6 earthquakes at R=30 km). The large variabilities in PID predictions is mainly due to the very low PID values and the sensitivity of this response parameter to higher modes and ground motion profiles at high intensity level (Lee and Mosalam, 2005).

Flexural Demand

Figure D.7 gives the maximum and M±SD predicted peak wall overturning moment (PWOM) responses to all input motions for 1% and 2% damping. Predictions are compared to the probable (yield) moment resistance envelope (M$_{pu}$) of the cantilever wall structure and the design moment demand for this structure, as suggested in the CAC 1995.

Figure D.7 shows that maximum PWOM predictions for 1% damping exceed the probable moment resistance envelope at the base of the wall and at storey 8, as previously indicated in Table D.4. For 2% damping, Figure D.7 shows that the maximum predicted PWOMs are lower than this envelope. Consequently, no plastic hinge is predicted in the wall at this damping level. However, this figure shows that the maximum predicted PWOMs for 2% damping exceed the design moment demand between storeys 8 and 11 inclusive. This observation does not apply to the the mean PWOM predictions. Actually the latter are lower than the design moment demand over all storeys. For 1% damping, both maximum and M+SD PWOM predictions exceed the design moment demand at storeys above wall mid-height.

The previous results show that the linear probable moment demand suggested in the CAC 1995 underestimates the predicted probable flexural demand at design level on the
cantilever wall structure at storeys above wall mid-height. Consequently, this linear demand was not sufficient to preclude the formation of a plastic hinge above the wall base for 1% damping. The large predicted PWOMs above wall mid-height are mainly due to the significant contribution of the second lateral mode of vibration, as shown in Figure 6.4.

Figure D.7 shows the capacity moment demand for the cantilever wall structure resulting from the application of the new capacity design provisions of the CSA A23.3-04. For comparison purposes, this demand is based on a probable rather than factored resistance, as specified in the CSA A23.3-04. Figure D.7 shows that this capacity moment demand gives a conservative estimate of the predicted flexural demand near wall mid-height and therefore, would have prevented the formation of the plastic hinge at this location for 1% damping.

Shear Demand

Figure D.8 gives the maximum and M±SD predicted peak wall shear force (PWSF) responses to all input motions for 1% and 2% damping. Predictions are compared to the factored shear resistance envelope \( (V_r) \) of the cantilever wall structure and the design shear force demand for this structure. The latter envelope is the probable shear force demand \( (V'_p) \) and was determined with Equation (3.8) of this document. The CSA A23.3-04 requires that \( V_r \) be not less than \( V'_p \). Although this requirement was satisfied at design stage, Figure D.8 shows that the predicted (probable) shear force demand exceeds \( V_r \) at some storeys. Nevertheless, the shape of the predicted shear demand over building height is very similar to that of \( V'_p \), which is essentially the shape of the SRSS shear force pattern, as shown in Figure 6.4.

For both damping levels, Figure D.8 shows that maximum PWSF predictions are significantly larger than \( V_r \). At wall base, the maximum predicted PWSF is about 35% and 20% greater than the factored base shear resistance (or design base shear) of 12600 kN for 1% and 2% damping, respectively. This was expected based on the inelastic analysis results presented in Section 7.1.3. Above storey 4, maximum PWSF predictions exceed \( V_r \) by as much as 30% and 20% for 1% and 2% damping, respectively. Consequently, maximum PWSF predictions are considerably larger than \( V'_p \). The major differences between the maximum PWSF predictions and \( V'_p \) are at storeys above wall mid-height. These dif-
ferences can be as large as 125% and 75% for 1% and 2% damping, respectively. Although they are lower, M+SD PWSF predictions are in general not so different from maximum PWSF predictions. Figure D.8 indicates that mean PWSF predictions do not exceed \( V_r \) and \( V_p \) as much as the maximum predictions for both damping levels. Actually mean PWSF predictions for 2% damping are almost the same as \( V_p \), with a little mismatch at storeys above wall mid-height. In spite of this, \( V_p \) is mostly within or equal to the M±SD PWSF predictions over wall height for this damping level. For 1% damping, however, a large mismatch between \( V_p \) and M±SD PWSF predictions is observed in Figure D.8. Moreover, the mean predicted PWSF at the base of the wall exceeds by almost 15% the design base shear force. The above predictions indicate that \( V_p \) considerably underestimates at wall base and above wall mid-height the shear force demand on the cantilever wall structure. Although the base shear force demand exceeds \( V_r \), it is still lower than the probable base shear resistance of the wall, which is about 18000 kN.

It is of interest to note that the large PWSF predictions at wall base and above wall mid-height are not only due to the formation of plastic hinges at these locations. Actually no hinging is predicted for 2% damping and still PWSFs considerably larger than \( V_p \) are predicted. This suggests that the large shear force predictions do not mainly result from the linear shear assumption used for modelling the behavior in shear of the wall. Large PWSF predictions do not depend either on the type of seismic events (short-period or long-period events) used as input motions, as shown in Figure D.9. This suggests that the large PWSF predictions are due to higher mode effects. The second lateral mode of vibration must significantly contribute to these effects since it governs the wall shear force response, as shown in Figure 6.4.

Discussion

The dynamic response results obtained from the cantilever wall model NSW-R bring out important findings regarding the seismic performance and design of the cantilever wall structure. Prior to draw any conclusions, the dynamic response results obtained from the model of the complete building structure in the cantilever wall direction must be considered. These results are presented in the following section.
8.2.2 Building in the Cantilever Wall Direction

In this section, the dynamic peak wall response predictions obtained from the building model NSB-R are compared to those obtained from the cantilever wall model NSW-R. This comparison allows to assess the effect of the added stiffness from structural components not part of the wall system on the cantilever wall response.

Overall Drift and Plastic Hinges

Table D.6 gives the maximum roof drift ($\Delta_{max}$), plastic hinges and associated maximum curvature ductility ratios ($\mu_{\phi}^{max}$) predicted from the building model for 1% and 2% damping. A comparison between this table and Table D.4 shows, as expected, that the maximum roof drifts predicted from the building model are in general slightly lower than those predicted from the isolated wall model. Considering the very low predicted maximum drift values and the significant dispersion of these values (COV $\approx 0.30$), maximum drift predictions obtained from the building model match reasonably well design values.

Table D.6 indicates that the building model predicts plastic hinges only in the wall and only under the long-period ground motions (MTL-M7 records), as expected. All predicted hinges are located at the base of the wall. Moreover, hinge and $\mu_{\phi}^{max}$ predictions are the same for both considered damping levels. Previous predictions contrast with those obtained from the isolated wall model. As shown in Table D.4, the isolated wall model predicts hinges only for 1% damping and one of these hinges is located at the base of storey 8. It appears then that the hinge prediction at this upper storey is more a modelling issue than a design issue. Moreover, the isolated wall model appears to be more sensitive to damping than the building model with regard to hinge predictions. This means that the added stiffness from structural components not part of the wall system has a significant influence on the hinge predictions in the wall. This suggests that this stiffness be accounted for when predicting plastic hinges in a slender ductile concrete cantilever wall system. Neglecting this stiffness can lead to inadequate hinge predictions and therefore to a requirement of unnecessarily providing a detailing for plastic hinging at predicted hinge locations. Nevertheless, for simple wall building structures, such as the studied one in this work, an isolated wall system model can provide a good estimation of the likely plastic hinges that may form in the system if it experiences seismic events more
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severe than that used for design.

Interstorey Drift

Figure D.10 gives the maximum and mean PID responses to all input motions predicted from the isolated wall and building models for 1% and 2% damping. Predictions are compared to the design interstorey drifts, which were determined from the linear modal response spectrum method specified in the NBCC 2005, using the SRSS combination. Figure D.10 shows that PID predictions obtained from the isolated wall and building models are mostly the same below approximately wall mid-height. Above the latter, PIDs predicted from the building model are somewhat lower than those predicted from the isolated wall model, except for the PIDs predicted at storey 8 for 1% damping. For this case, the PID predicted from the building model is about 50% lower than that predicted from the isolated wall model since the building model does not predict any plastic hinge at this location. Figure D.10 indicates that the design interstorey drifts underestimate the PID predictions obtained from the building model. The underestimation is considerable in comparison with the maximum PID predictions but relatively small compared to mean PID predictions. Once again, maximum PID predictions are due to a single ground motion record, the MTL2-M7 record. This explains the large difference between maximum and mean PID predictions. Table D.5 shows that the variability in PID predictions obtained from the building model is almost the same as that in PID predictions obtained from the isolated wall model.

Flexural demand

Figure D.11 gives the maximum and mean PWOM responses to all input motions predicted from the isolated wall and building models for 1% and 2% damping. Predictions are compared to the probable (yield) moment resistance envelope ($M_{pew}$) of the cantilever wall structure and the design moment demand for this structure.

For 2% damping, Figure D.11 shows that PWOM predictions obtained from the building model are almost the same as those obtained from the isolated wall model, except for the maximum predictions at the base of the wall. At this location, the building model
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predicts a plastic hinge in the wall but not the isolated wall model. For 1% damping, mean PWOMs predicted from the isolated wall and building models are mostly the same. However, for this damping level, maximum PWOMs near wall mid-height predicted from the building model are significantly lower than those predicted from the isolated wall model. They are lower enough to avoid the formation of plastic hinges above the base of the wall but not enough to match or be lower than the design moment demand. Actually, for both considered damping levels, the design demand somewhat underestimates the maximum PWOMs near wall mid-height predicted from the building model but matches quite well the mean PWOMs at this location predicted from this model. Table D.5 shows that the variability in PWOM predictions obtained from the building model is almost the same as that in PWOM predictions obtained from the isolated wall model.

The above results show that the added stiffness from structural components not part of the wall system has a significant effect on the maximum predicted flexural demand on the cantilever wall structure when a low (1%) but realistic damping level is used for analysis.

Shear demand

Figure D.12 gives the maximum and mean PWSF responses to all input motions predicted from the isolated wall and building models for 1% and 2% damping. Predictions are compared to the factored shear resistance envelope \( V_r \) of the cantilever wall structure and the design shear force demand \( V_d \) for this structure.

Figure D.12 shows that PWSF predictions obtained from the isolated wall and building models are practically speaking the same. In comparison with the peak predictions obtained from the isolated wall model, the building model predicts a little increase of the maximum PWSF at wall base but at the same time, a little decrease of the mean PWSF at this location. However, these differences in predictions are negligible. Thus, the wall shear force demand predicted from the building model still considerably exceeds \( V_d \) at wall base and at storeys above wall mid-height. Table D.5 shows that the variability in PWSF predictions obtained from the building model is not different from that in PWSF predictions obtained from the isolated wall model.
The above results show that the added stiffness from structural components not part of the wall system has very little effect on the predicted shear force demand on the cantilever wall structure. However, it has a significant one on the maximum predicted flexural demand, and therefore on the plastic hinge predictions, when a low but realistic damping level is considered for analysis.

Discussion

Based on Tables 2.1 and 2.2, the overall estimated damage state of the cantilever wall structure at design level corresponds to the performance level "operational" rather than "near collapse", which is the performance level expected for the seismic event (2500-year return period) used for design. This high level of performance of this structure is due mainly to its relatively great flexural overstrength, which results from the excess strength arising from the required minimum reinforcement.

In spite of the fact that the added stiffness from structural components not part of the wall system has either negligible or beneficial effects on the predictions, it is important to note that the predictions do not include torsional effects. This means that the seismic demands on the wall at design level could be greater than that predicted in this project.

The predictions obtained from the isolated wall and building models indicate that higher mode effects have a considerable influence on the seismic demand at design level on the cantilever wall structure. The second lateral mode appears to be the higher mode of vibration contributing the most to these effects. The dynamic amplification due to higher mode effects is such that the predicted seismic demand significantly exceeds the seismic demand used for design, which was determined from the linear modal response spectrum method specified in the NBCC 2005, using the SRSS combination, and a capacity design approach. However, no such excess would have been predicted in flexure if the capacity moment demand resulting from the application of the new capacity design provisions of the CSA A23.3-04 had been used for design. This demand, unlike the linear moment demand suggested in the CAC 1995, is an amplified version of the factored moment demand \( M_f \). The shape of the latter along the wall height is then accounted for in the former. This means that the large flexural demand near wall mid-height due to higher mode effects can be taken into account by the new capacity moment demand prescribed by the CSA A23.3-
04, as long as the factored moment demand is obtained from linear dynamic analysis.

The above results bring out two possible reasons that can explain the mismatch between the seismic demand used for design and the predicted one: either higher mode effects were underestimated when determining the NBCC seismic design demand or predictions obtained from the inelastic time-history dynamic analysis overestimate these effects. The most reasonable choice would be the second one because of the various uncertainties inherent to the inelastic time-history dynamic analysis method. However, it is believed that this is in part true and that the first reason mostly contributes to the large excess of the predicted seismic demand.

Various reasons may explain why higher mode effects were underestimated at design stage. One could attribute this underestimation to the fact that the NBCC seismic design demand was determined from a linear elastic analysis or that the effective flexural stiffness used for design was inadequate. The influence of these factors is estimated to be negligible since no to very little yielding is predicted in the studied cantilever wall and the elastic modal characteristics of the wall model used for design (see Table 6.3) are almost the same as those of the wall model used for the inelastic time-history analysis (see Table B.15). The underestimation of the higher mode effects is believed to come mainly from the inadequateness of the spectral response acceleration \( S(T) \) or \( S_a(T) \), see Eq. 3.4) used for design with regard to the predicted seismic behavior of the cantilever wall.

The design spectral response acceleration \( S(T) \) is based on an equivalent viscous damping ratio of 5%. This common damping value is intended to account for the total energy dissipation in the structural resisting system, including the plastic part, since \( S(T) \) is essentially a uniform response spectrum (UHS) of an elastic SDOF system. However, very little plastic deformation is predicted in the studied cantilever wall. In addition, inelastic wall responses to ground motions are computed for 1% and 2% damping, which correspond to damping in the undamaged structure. This means that the total energy dissipation assumed for design is greater than that predicted from the inelastic analysis and therefore, that \( S(T) \) is less than the spectral acceleration response predicted by the inelastic wall model during the time-history analysis. As the predicted behavior of the cantilever wall is mostly elastic, this difference can be estimated by comparing \( S(T) \) to the 1%- and 2%-damped pseudo-spectral acceleration response spectra of the ground motions used as input motions for analysis. Figure D.13 illustrates this comparison for the
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MTL2-M6 and MTL2-M7 records. This figure shows not much difference between spectral accelerations for periods in the range of the fundamental lateral period of the elastic wall model (1.74 s) used for design and that of the inelastic wall model (1.59 s) used for analysis. However, for periods in the range of higher modes, the spectral accelerations for 1% and 2% damping are considerably greater than those of the design spectrum. Although they are not presented in this document, the same observations can be made for the other ground motions used as input motion. Previous results clearly explain the larger higher mode contribution, particularly the second lateral mode contribution, in the predicted seismic demand than in the design seismic demand.

Figure D.13 indicates that the match between the design and predicted seismic demands should be relatively good if the inelastic analysis is carried out with 5% damping. The results shown in Figures D.14 and D.15 confirm quite well this statement. This is not a surprising result as predictions indicate that the wall responds elastically for 5% damping. The mismatch between the design and maximum predicted shear force demands at upper storeys suggests that an effective flexural stiffness larger than 0.71g, but less than Ig, could have been used at this location for design. The match of the linear probable moment demand with the maximum moment predictions above wall mid-height is very likely due to the large flexural overstrength of the wall structure. Actually the work of Tremblay et al. (2001) suggests that the flexural demand at this location for 5% damping can be considerably greater than the linear probable moment demand when a 12-storey ductile concrete cantilever wall has little flexural overstrength.

The above results show that the 5% damping on which are based the NBCC spectral response accelerations (Sa(T)) may be inadequate for the seismic design, based on the linear modal response spectrum method, of reinforced concrete (RC) cantilever wall structures where the seismic response is mostly elastic and dominated by higher modes. This suggests that Sa(T) values be also determined for lower damping values and be related to the expected seismic behavior of the designed wall structure.

A suitable application of the previous suggestion requires a good estimation of the expected seismic behavior of the designed wall structure. The Me factor specified in the NBCC 2005 provides a reasonable estimation on the sensibility of a RC shear wall structure to higher mode effects (see Table 3.2 of this document). The new displacement-based provisions of the CSA A23.3-04 for ductility assessment of such structures (see
Section 2.2.2 of this document) also provide a good tool to estimate the expected wall behavior (elastic, inelastic...) in flexure at design level. However, the quantitative inelastic rotational demands determined from these provisions do not easily provide to practitioners a qualitative assessment on the flexural behavior to be expected for the predicted ductility demand. This suggests that a chart relating inelastic rotational demand to expected wall behavior be established and included in the CSA A23.3 Standard. This chart would provide a rational verification on the selected $R_d$ value used for design. However, this verification would come after a preliminary detailing of the RC wall structure.

8.2.3 Important Findings for the Cantilever Wall Structure

The dynamic analysis results presented in Sections 8.2.1 and 8.2.2 lead to the following important findings with regard to the seismic performance of the studied cantilever wall structure at design level, its seismic design and its structural modelling:

1. The overall seismic performance of the wall is much better than that expected for the seismic event (2500-year return period) used for design. This is essentially due to the relatively great flexural overstrength of the wall. This suggests the necessity of minimizing the excess strength in the expected hinging regions;

2. The interstorey drift demand is considerably underestimated by the design interstorey drifts determined from the linear modal dynamic analysis method specified in the NBCC 2005;

3. The flexural demand on the wall at storeys above wall mid-height is slightly underestimated by the linear probable moment demand suggested in the CAC 1995 for seismic design. The predicted demand, however, is conservatively estimated by the capacity moment demand resulting from the application of the new capacity design provisions of the CSA A23.3-04. Nevertheless, the formation of a plastic hinge may be expected near wall mid-height for seismic events much more severe than that used for design;

4. The shear force demand on the wall is considerably underestimated at wall base and at storeys above wall mid-height by the design (probable) shear force demand
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... determined with Equation (3.8). As a consequence, the shear strength requirement prescribed by the CSA A23.3-04 is not satisfied, that is, the probable shear demand exceeds the factored shear resistance ($V_t$);

5. The added stiffness from structural components not part of the wall system has a significant effect on the maximum predicted flexural demand on the wall but not on its predicted shear force demand. Neglecting this stiffness misleads hinge predictions. This suggests that this stiffness be accounted for when predicting the seismic flexural demand on a slender ductile concrete cantilever wall system;

6. The significant underestimation of the predicted seismic demand by that used for design is mainly due to the inadequateness of the design spectral response acceleration with regard to the predicted seismic behavior (mostly elastic) of the cantilever wall. The 5%-damped spectral response acceleration, $S_\alpha(T)$, used for design overestimates the total energy dissipation in the predicted cantilever wall response and therefore, underestimates the dynamic amplification on the seismic response due to higher mode effects. This suggests that the NBCC spectral response acceleration may be inadequate for the seismic design, based on the linear modal dynamic analysis method, of RC cantilever wall structures where the seismic response is mostly elastic and dominated by higher modes.

8.2.4 Coupled Wall

The predictions presented in this section were obtained from the coupled wall model EWW-R.

Overall Drift and Plastic Hinges

Table D.7 gives the maximum roof drift ($\Delta_{max}$), plastic hinges and associated maximum curvature ductility ratios ($\mu^{max}_\phi$) predicted for 1% and 2% damping. This table shows that all predicted maximum roof drifts are almost the same as the design lateral drift of 0.18%, for both damping levels. It is of interest to note that the dispersion in maximum roof drift predictions obtained from the coupled wall model (COV ≈ 0.20) is less than that in
the maximum roof drift predictions obtained from the cantilever wall model or the N-S building model (COV ≈ 0.30, see Tables D.4 and D.6). The coupled wall model predicts plastic hinges only in coupling beams. The predicted \( \mu_{\theta}^{\text{max}} \) in these hinges range between 1.4 and 2.5. These predictions match quite well those obtained from the pushover analysis for a probable resistance, as shown in Table D.2. As expected, the predicted curvature ductility demand in beams is very low and is well below the ultimate curvature ductility ratio of 10. Figure D.16 illustrates the distribution over wall height of the maximum and M±SD peak curvature ductility ratios predicted in coupling beams for 1% and 2% damping. This figure indicates that the maximum and M+SD predictions match quite well and that there is no significant differences between maximum and mean peak predictions as the dispersion in peak predictions is low (COV ≈ 0.15, see Table D.5). Moreover, it shows that peak predictions are almost the same for both considered damping levels.

**Interstorey Drift**

Figure D.17 gives the maximum and M±SD predicted PID responses to all input motions for 1% and 2% damping. Predictions are compared to design interstorey drifts, which were determined from the linear modal response spectrum method specified in the NBCC 2005, using the SRSS combination. This figure shows that the difference between the maximum and mean PID predictions is not significant. That is because the variability in PID predictions is not much (COV ≈ 0.20), as given in Table D.5. This differs from the results obtained for the cantilever wall (see Figure D.5). Moreover, Figure D.17 shows that PID predictions are almost the same for both considered damping levels. Maximum PID predictions are due to the MTL-M7 records. Considering the very low predicted PID values, Figure D.17 shows in general that PID predictions match relatively well design interstorey drifts.

**Flexural demand**

Figure D.18 gives the maximum and M±SD predicted peak wall bending moment (PWBM) responses to all input motions for 1% and 2% damping. Predictions are the bending moments acting on one C-shaped wall when the latter acts as a tension or compression wall. Predictions are compared to the probable (yield) moment resistance en-
velopes \( (M_{pw}) \) of one C-shaped wall structure and the design moment demand for this structure. The minimum and maximum expected probable moment resistances are presented in Figure D.18 for the tension and compression wall, respectively. These resistances were determined by accounting for the maximum probable earthquake-induced axial force acting on a C-shaped wall. This force was estimated with Equation (3.9) of this document. The design demand is the linear probable moment demand, as suggested in the CAC 1995, and is that for the compression wall, as it is the most conservative. As shown in Figure D.18, this demand assumes that the plastic hinge region in the wall is over the two first storeys from the base of the wall. This height is about 70% greater than the minimum assumed plastic hinge height of \( 1.5\ell_w \) required by the CSA A23.3-04, where \( \ell_w \) in this case is the horizontal length of one C-shaped wall in the E-W direction.

Figure D.18 shows that maximum PWBM predictions are not so different from the M+SD predictions and are not in general considerably greater than the mean peak predictions. That is because the variability in peak predictions is small \( (COV \approx 0.12, \text{ see Table D.5}) \), as illustrated in Figure D.19. The maximum and mean PWBM predictions are mostly lower than the design demand. Above storey 7, the mean PWBM predictions match this demand while the maximum ones exceed it. The most significant excesses occur when one of the two C-shaped walls acts as a tension wall. It is noted that these excesses are mostly the same for both considered damping levels. In general, it is observed that the predicted flexural demand on a C-shaped wall is conservatively to fairly estimated by the design moment demand.

Figure D.18 shows the capacity moment demand for the compression wall resulting from the application of the new capacity design provisions of the CSA A23.3-04. For comparison purposes, this demand is based on a probable rather than factored resistance, as specified in the CSA A23.3-04. Figure D.7 shows that this capacity moment demand matches or exceeds the predicted flexural demand over the entire wall height for both considered damping levels. In addition, the use of this demand for design would have resulted to a greater flexural strength of the C-shaped walls between storeys 6 and 10. Thus, the capacity moment demand prescribed by the CSA A23.3-04 provides a better and more conservative estimate of the flexural demand to be expected on the C-shaped walls at design level than the linear probable moment demand suggested in the CAC 1995.
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Shear demand

Figure D.20 gives the maximum and M+SD predicted PWSF responses to all input motions for 1% and 2% damping. Predictions are compared to the factored shear resistance envelope \( V_r \) of one C-shaped wall structure and the design shear force demand for this structure. This demand is the probable shear force demand \( V_p \) and was determined with Equation (3.8) of this document. The CSA A23.3-04 requires that \( V_r \) be not less than \( V_p \). Figure D.20 shows that this requirement is satisfied for the design shear forces but not for the predicted (probable) wall shear forces at the base of the wall. This figure also shows that the shear reinforcement required for the assumed plastic hinge region is maintained over the first four storeys from the base of the wall. Above these storeys, the required minimum shear reinforcement is provided.

Figure D.20 indicates that the difference between maximum and mean PWSF predictions is little, except at wall base and at storeys above wall mid-height. At these locations, maximum PWSF predictions are even significantly greater than M+SD predictions. These large predictions are not due to a single ground motion record but rather to different records, as show in Figure D.21. This means that maximum PWSF predictions are more likely. Figure D.20 indicates that the maximum PWSF predictions at the base of the wall exceed \( V_r \) (or \( V_p \)) by about 50% and 45% for 1% and 2% damping, respectively. For the mean PWSF predictions at wall base, these differences drop to 20% for both damping levels. It is of interest to note that these significant exceedances occur when one of the two C-shaped wall acts as a compression wall. This base shear demand increase on the compression wall was expected, as illustrated in Figure D.4, for the predicted overall deformation range. However, despite this increase, the base shear demand should not have exceeded \( V_r \). Above about wall mid-height, Figure D.20 shows that maximum and even M+SD PWSF predictions are considerably underestimated by \( V_p \) but not the mean peak predictions. Nevertheless, the latter are somewhat larger than \( V_p \).

The above results indicate that the predicted shear force demand on each C-Shaped wall is significantly underestimated at wall base and at storeys above wall mid-height by the design (probable) shear force demand determined with Equation (3.8). Once again, this result appears to be due to an underestimation of the higher mode effects by the NBCC seismic design demand, which was determined from a linear modal dynamic analysis.
It is noted that the large base shear force predictions in walls should not be due to the linear shear assumption used for modelling the shear behavior of the wall members since walls do not yield (Derecho et al., 1979).

Discussion

Based on Tables 2.1 and 2.2, the overall estimated damage state of the coupled wall structure at design level corresponds to the performance level "operational" rather than "near collapse", which is the performance level expected for the seismic event (2500-year return period) used for design. This high level of performance of this structure is due mainly to its relatively great flexural overstrength, which results from the excess arising from the required minimum reinforcement in the wall members.

The analysis results in the coupled wall direction show that the predicted seismic demand is mostly as estimated by the capacity demand used for design. However, a significant underestimation of the predicted shear force demand in walls by that used for design is observed at wall base and at storeys above wall mid-height. This is due to the higher sensitivity of the shear demand to higher mode effects. Nevertheless, differences between the predicted and design shear force demands are less significant for the coupled wall than for the cantilever wall. This means that the total energy dissipation predicted in the coupled wall is not so different from that assumed for design, which corresponds to an equivalent viscous damping of 5% ($S_a(T)$). This differs from the cantilever wall direction where the energy dissipation is mainly due to the 1% and 2% damping used for analysis, as very little inelastic action is predicted in the wall at design level. For the coupled wall, the predicted inelastic action in the coupling beams contributes to increase the energy dissipation in the wall to a level similar to that assumed for design, even if the predicted beam ductility demand is low. This suggests that the NBCC spectral response acceleration is reasonably adequate for the seismic design of mid-rise RC coupled wall systems, even if the expected beam ductility demand at design level is relatively low.
8.2.5 Important Findings for the Coupled Wall Structure

The dynamic analysis results presented in Section 8.2.4 lead to the following important findings with regard to the seismic performance of the coupled wall structure at design level and its seismic design:

1. The overall seismic performance of the coupled wall is much better than that expected for the seismic event (2500-year return period) used for design. This is essentially due to the relatively great flexural overstrength of the wall system. This suggests the necessity of minimizing the excess strength in the expected hinging regions;

2. The interstorey drift demand is mostly as estimated by the design interstorey drifts determined from the linear modal dynamic analysis method specified in the NBCC 2005;

3. The flexural demand on the walls at storeys above wall mid-height is slightly underestimated by the linear probable moment demand suggested in the CAC 1995. The capacity moment demand prescribed by the CSA A23.3-04 provides a better and more conservative estimate of the flexural demand on the walls;

4. The shear force demand on the walls is significantly underestimated at wall base and at storeys above wall mid-height by the design (probable) shear force demand determined with Equation (3.8). Consequently, the shear strength requirement prescribed by the CSA A23.3-04 for walls is not satisfied, that is, the probable shear demand exceeds the factored shear resistance $V_r$;

5. Despite an underestimation of higher mode effects by the design shear demand, the NBCC spectral response acceleration ($S_a(T)$) appears to be reasonably adequate for the seismic design of mid-rise RC coupled wall systems, even if the expected beam ductility demand at design level is relatively low.
Chapter 9

Conclusion

This research project has assessed the seismic performance a ductile concrete core wall system used as a Seismic Force Resisting System (SFRS) for a 12-storey concrete building designed according to the NBCC 2005 and the CSA A23.3-04. This shear wall system consists of a cantilever wall system in one direction and a coupled wall system in the orthogonal direction. The assessment is based on two-dimensional inelastic pushover and time-history dynamic analyses.

From the work performed in this project, the following important conclusions can be drawn, with regard to:

Structural Modelling for Inelastic Analysis

1. The concrete tension-stiffening effect has a significant influence on the cantilever wall response predicted from inelastic pushover and time-history dynamic analyses, particularly when this response is mostly elastic. When it is accounted for, the concrete tension-stiffening effect produces more reliable and numerically stable dynamic predictions, especially for hinge and shear force predictions in the wall structure;

2. Wall models using general two-component elements generally provide better predic-
tions than those using one-component beam elements, especially when a fine element discretization is used over the wall storeys;

3. The added stiffness from structural components not part of the wall system has a significant effect on the maximum predicted flexural demand on the cantilever wall but not on its predicted shear force demand. Neglecting this stiffness may mislead hinge predictions.

Seismic Performance and Design

1. The overall seismic performance of the core wall system is much better than that expected for the 2500-year return period event used for seismic design. This is essentially due to the relatively great flexural overstrength of the wall system. This overstrength is largely due to the large excess strength arising from the required minimum reinforcement in the assumed base plastic hinge region of the wall;

2. Very low inelastic rotational demands are expected in the respective energy-dissipating mechanisms of the wall system, though this system is designed and detailed to be ductile \((R_d > 3)\);

3. The design interstorey drifts determined from the linear modal dynamic analysis method specified in the NBCC 2005 considerably underestimate the interstorey drift demand at design level in the cantilever wall direction;

4. The linear probable moment demand suggested in the CAC 1995 slightly underestimates the flexural demand (at design level) on the cantilever wall system at storeys above wall mid-height. The capacity moment demand resulting from the application of the new capacity design provisions of the CSA A23.3-04 provides a better and more conservative estimate of the flexural demand on the wall members in either principal direction. Nevertheless, the formation of a plastic hinge in the cantilever wall structure may be expected near wall mid-height for seismic events much more severe than that used for design;

5. The design (probable) shear force demand determined with Equation (3.8) considerably underestimates the shear force demand (at design level) at wall base and above wall mid-height, especially in the cantilever wall direction. Consequently, the shear
strength requirement prescribed by the CSA A23.3-04 is not satisfied, that is, the probable shear demand exceeds the factored shear resistance $V_r$ of the wall members;

6. The significant underestimation of the seismic demand on the cantilever wall system by that used for design is mainly due to the inadequateness of the NBCC design spectral response acceleration with regard to the seismic behavior (mostly elastic) at design level of the cantilever wall system. The 5%-damped spectral response acceleration, $S_a(T)$, used for design overestimates the total energy dissipation in the predicted cantilever wall response and therefore, underestimates the dynamic amplification on the seismic response due to higher mode effects. This suggests that the NBCC spectral response acceleration may be inadequate for the seismic design, based on the linear modal dynamic analysis method, of RC cantilever wall systems where the seismic response is mostly elastic and dominated by higher modes.

9.1 Recommendations

Although previous conclusions are based on a single typical shear wall building, they suggest that:

1. The concrete tension-stiffening effect and the added stiffness from structural components not part of the wall system be accounted for when assessing from inelastic analysis the seismic performance of ductile concrete shear wall systems;

2. The flexural overstrength of such systems be minimized by avoiding as much as possible the excess strength in the expected hinging regions.

3. The linear probable moment demand suggested in the CAC 1995 be not used for the seismic design of ductile concrete shear wall systems where the seismic response is dominated by higher modes and the flexural overstrength is particularly low;

4. The $S_a(T)$ values be also determined for lower damping values and be related to the expected seismic behavior of the designed wall structure.
9.2 Future Research

Based on the previous conclusions and recommendations, the following future researches are suggested:

1. Studying the influence of the concrete tension stiffening effect on the numerical predictions in different ductile concrete shear wall systems;

2. Validating experimentally the dynamic amplification on strength demand due to higher mode effects numerically predicted in such systems;

3. Developing a design method in a Canadian code perspective to suitably estimate the shear force demand in ductile concrete shear wall systems;

4. Developing a chart for the CSA A23.3 Standard relating the inelastic rotational demand determined with the new provisions for ductility assessment to the seismic wall behavior to be expected for the predicted ductility demand.
Appendix A

Detail Drawings of the Structural Components
Figure A.2: Reinforcement details of a typical spandrel beam
APPENDIX A. DETAIL DRAWINGS OF THE STRUCTURAL COMPONENTS  143

Interior Columns

Storeys 1 to 3
12 No. 25
No. 10 @ 250 mm

Storeys 4 to 6
12 No. 20
No. 10 @ 225 mm

Storeys 7 to 9
8 No. 20
No. 10 @ 200 mm

Storeys 10 to 12
8 No. 20
No. 10 @ 175 mm

Side Columns

Storeys 1 to 3
8 No. 25
No. 10 @ 200 mm

Storeys 4 to 6
8 No. 20
No. 10 @ 200 mm

Storeys 7 to 9
6 No. 20
No. 10 @ 175 mm

Storeys 10 to 12
6 No. 20
No. 10 @ 150 mm

Corner Columns

Storeys 1 to 12
8 No. 20
No. 10 @ 175 mm

For all columns
Clear concrete cover from side face to ties: 30 mm

Figure A.3: Reinforcement details of columns
Figure A.4: Reinforcement details of typical diagonally reinforced coupling beams
Figure A.5: Reinforcement details of one C-shaped wall - Section view
Figure A.6: Reinforcement details of one C-shaped wall - Elevation view
Appendix B

Structural Models for Inelastic Analysis
Table B.1: Elastic material and beam section properties used in RU-AUMOKO for analysis

<table>
<thead>
<tr>
<th>Member</th>
<th>$E_c$</th>
<th>$G_c^{(1)}$</th>
<th>$A$</th>
<th>$A_y$</th>
<th>$I$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[m$^2$]</td>
<td>[m$^2$]</td>
<td>[m$^4$]</td>
</tr>
<tr>
<td>C-shaped wall$^{(2)}$</td>
<td>25000</td>
<td>10420</td>
<td>3.416</td>
<td>2.20</td>
<td>4.9735</td>
</tr>
<tr>
<td>Coupling beam$^{(3)}$</td>
<td>25000</td>
<td>10420</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Floor levels 2 to 7</td>
<td></td>
<td></td>
<td>0.80</td>
<td>0.28</td>
<td>0.0133</td>
</tr>
<tr>
<td>Floor levels 8 to 10</td>
<td></td>
<td></td>
<td>0.80</td>
<td>0.24</td>
<td>0.0100</td>
</tr>
<tr>
<td>Floor levels 11 to penthouse roof</td>
<td></td>
<td></td>
<td>0.80</td>
<td>0.16</td>
<td>0.0067</td>
</tr>
<tr>
<td>Cantilever wall$^{(4)}$</td>
<td>25000</td>
<td>10420</td>
<td>9.76</td>
<td>5.12</td>
<td>59.3</td>
</tr>
</tbody>
</table>

1- $G_c = E_c/2(1+\nu)$ with $\nu = 0.2$.

2- The $A$ value is equal to 0.7 $A_y$. However, for pushover analysis, $A$ is taken as 0.6 $A_y$ for the tension wall and $A_y$ for the compression wall. Reduction factors are calculated with Equation (3.6). The $A_y$ value is equal to 5/6 of the combined cross-sectional areas of the flanges of the C-shaped section (Wilson and Habibullah, 1992). This value is approximately equal to 0.45 $A_y$.

3- Elastic section properties are for two coupling beams. The $A$ value is equal to $A_y$. The $A_y$ and $I$ values account for cracking as follows: $A_y = 0.35$, 0.30 and 0.20 $A_y$ and $I = 0.20$, 0.15 and 0.10 $I_y$ for beams located at floor levels 2-7, 8-10 and 11-penthouse roof, respectively.

4- Elastic section properties are for two C-shaped walls only. Coupling beams are neglected. The $A$ and $I$ values are equal to $A_y$ and $I_y$, respectively. The $A_y$ value is equal to the cross-sectional area of the webs of the wall section (Wilson and Habibullah, 1992). This value is approximately equal to 0.52 $A_y$.

Table B.2: Material properties used in MNPHi for (unconfined) concrete (see Fig. B.1(a))

<table>
<thead>
<tr>
<th>$E_c$</th>
<th>$f'_c$</th>
<th>$\varepsilon'_c$</th>
<th>$f_{cr}$</th>
<th>$\varepsilon_{c,max}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[mm/mm]</td>
<td>[MPa]</td>
<td>[mm/mm]</td>
</tr>
<tr>
<td>25000</td>
<td>30</td>
<td>0.002</td>
<td>3.3</td>
<td>0.0035</td>
</tr>
</tbody>
</table>

1- The flexural yield strength of a section is determined with the equivalent rectangular concrete stress distribution specified in the CSA A23.3-04.

2- The tensile stress-strain relationship used to compute the bending moment-curvature response for the UB case is that proposed by Vecchio and Collins (1986).
Figure B.1: Material stress-strain relationships used in MNPHI

Table B.3: Material properties used in MNPHi for reinforcing steel (see Fig. B.1(b))

<table>
<thead>
<tr>
<th>$E_s$</th>
<th>$f_y$</th>
<th>$\varepsilon_{sh}$</th>
<th>$f_u$</th>
<th>$\varepsilon_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[mm/mm]</td>
<td>[MPa]</td>
<td>[mm/mm]</td>
</tr>
<tr>
<td>200000</td>
<td>400$^{(1)}$</td>
<td>0.01</td>
<td>600$^{(1,2)}$</td>
<td>0.1</td>
</tr>
</tbody>
</table>

1- $f_y = 500$ MPa ($= 1.25 \cdot 400$ MPa) and $f_u = 750$ MPa ($= 1.25 \cdot 600$ MPa) for a probable resistance.

2- The flexural yield strength of a section is determined with an elasto-perfectly plastic relationship (no strain hardening, $f_u = f_y$).
Table B.4: Seismic weights of the building used for analysis

<table>
<thead>
<tr>
<th>Floor level</th>
<th>Weight [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>8105</td>
</tr>
<tr>
<td>12</td>
<td>8279</td>
</tr>
<tr>
<td>11</td>
<td>8279</td>
</tr>
<tr>
<td>10</td>
<td>8310</td>
</tr>
<tr>
<td>9</td>
<td>8341</td>
</tr>
<tr>
<td>8</td>
<td>8341</td>
</tr>
<tr>
<td>7</td>
<td>8374</td>
</tr>
<tr>
<td>6</td>
<td>8408</td>
</tr>
<tr>
<td>5</td>
<td>8408</td>
</tr>
<tr>
<td>4</td>
<td>8432</td>
</tr>
<tr>
<td>3</td>
<td>8455</td>
</tr>
<tr>
<td>2</td>
<td>8738</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>100468</strong></td>
</tr>
</tbody>
</table>
Figure B.2: Different beam element discretizations of the cantilever wall
Figure B.3: 2D structural model NSW3-E of the cantilever wall used in EFICoS
## Table B.5: Multilayered beam element section properties used in EFiCoS

<table>
<thead>
<tr>
<th>Layer</th>
<th>Storeys 1-6</th>
<th>Storeys 7-13</th>
</tr>
</thead>
<tbody>
<tr>
<td>No.</td>
<td>Width [mm]</td>
<td>Thickness [mm]</td>
</tr>
<tr>
<td>1</td>
<td>6600</td>
<td>40</td>
</tr>
<tr>
<td>2</td>
<td>6600</td>
<td>25</td>
</tr>
<tr>
<td>3</td>
<td>6600</td>
<td>122.5</td>
</tr>
<tr>
<td>4</td>
<td>6600</td>
<td>25</td>
</tr>
<tr>
<td>5</td>
<td>6600</td>
<td>122.5</td>
</tr>
<tr>
<td>6</td>
<td>6600</td>
<td>25</td>
</tr>
<tr>
<td>7</td>
<td>6600</td>
<td>40</td>
</tr>
<tr>
<td>8</td>
<td>800</td>
<td>82.5</td>
</tr>
<tr>
<td>9</td>
<td>800</td>
<td>25</td>
</tr>
<tr>
<td>10</td>
<td>800</td>
<td>392.5</td>
</tr>
<tr>
<td>11</td>
<td>800</td>
<td>400</td>
</tr>
<tr>
<td>12</td>
<td>800</td>
<td>400</td>
</tr>
<tr>
<td>13</td>
<td>800</td>
<td>400</td>
</tr>
<tr>
<td>14</td>
<td>800</td>
<td>400</td>
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<tr>
<td>15</td>
<td>800</td>
<td>400</td>
</tr>
<tr>
<td>16</td>
<td>800</td>
<td>600</td>
</tr>
<tr>
<td>17</td>
<td>800</td>
<td>400</td>
</tr>
<tr>
<td>18</td>
<td>800</td>
<td>400</td>
</tr>
<tr>
<td>19</td>
<td>800</td>
<td>400</td>
</tr>
<tr>
<td>20</td>
<td>800</td>
<td>400</td>
</tr>
<tr>
<td>21</td>
<td>800</td>
<td>400</td>
</tr>
<tr>
<td>22</td>
<td>800</td>
<td>392.5</td>
</tr>
<tr>
<td>23</td>
<td>800</td>
<td>25</td>
</tr>
<tr>
<td>24</td>
<td>800</td>
<td>82.5</td>
</tr>
<tr>
<td>25</td>
<td>6600</td>
<td>40</td>
</tr>
<tr>
<td>26</td>
<td>6600</td>
<td>25</td>
</tr>
<tr>
<td>27</td>
<td>6600</td>
<td>122.5</td>
</tr>
<tr>
<td>28</td>
<td>6600</td>
<td>25</td>
</tr>
<tr>
<td>29</td>
<td>6600</td>
<td>122.5</td>
</tr>
<tr>
<td>30</td>
<td>6600</td>
<td>25</td>
</tr>
<tr>
<td>31</td>
<td>6600</td>
<td>40</td>
</tr>
</tbody>
</table>
Table B.6: Material properties used for the concrete damage model of La Borderie (1991)

<table>
<thead>
<tr>
<th>$E_c$</th>
<th>$Y_{01}$</th>
<th>$A_1$</th>
<th>$B_1$</th>
<th>$\beta_1$</th>
<th>$Y_{02}$</th>
<th>$A_2$</th>
<th>$B_2$</th>
<th>$\beta_2$</th>
<th>$\sigma_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>[MPa]</td>
<td>[Pa]</td>
<td>[Pa$^{-1}$]</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[MPa$^{-1}$]</td>
<td>[MPa]</td>
<td>[MPa]</td>
<td></td>
</tr>
<tr>
<td>26950</td>
<td>145$^{(2)}$</td>
<td>0.002$^{(1,2)}$</td>
<td>1.0</td>
<td>1.13</td>
<td>0.01</td>
<td>8.1</td>
<td>1.47</td>
<td>-34.3</td>
<td>2.3</td>
</tr>
</tbody>
</table>

1- Except for $A_1$, parameter values come from Légeron et al. (2005) and are for normal-strength concrete. Légeron et al. suggest $A_1 = 0.012$ Pa$^{-1}$.

2- Parameter $Y_{01}$ controls the peak tensile strength and parameters $A_1$ and $B_1$ control the post-peak behavior in tension (tension stiffening). To neglect the strength in tension of concrete, the $Y_{01}$ value is reduced to a value significantly lower than 145 Pa. To neglect tension stiffening, $A_1 = 1.0$ Pa$^{-1}$ is used, as suggested in Légeron et al. (2005). See Figure B.4.

![Graph](image)

Figure B.4: Effect of La Borderie's parameter $A_1$ on the pushover response of the EFiCoS model NSW3-E. All responses are for a nominal resistance.

Table B.7: Material properties used in EFiCoS for reinforcing steel

<table>
<thead>
<tr>
<th>$E_s$</th>
<th>$f_y$</th>
<th>$\epsilon_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>[MPa]</td>
<td>[MPa]</td>
<td>[mm/mm]</td>
</tr>
<tr>
<td>200000</td>
<td>400$^{(1)}$</td>
<td>0.05</td>
</tr>
</tbody>
</table>

1- $f_y = 500$ MPa ($= 1.25 \cdot 400$ MPa) for a probable resistance.
Figure B.5: Pushover predictions, for the LB case, obtained from the EFiCoS model NSW3-E and RUAUMOKO models NSW1-R, NSW2-R and NSW3-R meshed with (a) one-component (Giberson) beam elements, (b) general two-component beam elements. All responses are for a nominal resistance.
Figure B.6: Pushover predictions, for the UB case, obtained from the EFiCoS model NSW3-E and RUAUMOKO models NSW1-R, NSW2-R and NSW3-R meshed with (a) one-component (Giberson) beam elements, (b) general two-component beam elements. All responses are for a nominal resistance.
Table B.8: Base plastic hinge heights, at 2.5% lateral drift, predicted from RUAUMOKO and EFfCoS wall models for the LB case

<table>
<thead>
<tr>
<th>Wall model</th>
<th>Inelastic beam element</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>One-component</td>
</tr>
<tr>
<td>NSW1-R</td>
<td>Up to the base of S2 inclusive</td>
</tr>
<tr>
<td>NSW2-R</td>
<td>Up to the base of S2 exclusive</td>
</tr>
<tr>
<td>NSW3-R</td>
<td>Base of S1</td>
</tr>
<tr>
<td>NSW3-E</td>
<td>Yielding in tensile steel layers up to S4 exclusive</td>
</tr>
</tbody>
</table>

Table B.9: Base plastic hinge heights, at 2.5% lateral drift, predicted from RUAUMOKO and EFfCoS wall models for the UB case

<table>
<thead>
<tr>
<th>Wall model</th>
<th>Inelastic beam element</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>One-component</td>
</tr>
<tr>
<td>NSW1-R</td>
<td>Up to the base of S2 inclusive</td>
</tr>
<tr>
<td>NSW2-R</td>
<td>Up to the base of S2 inclusive</td>
</tr>
<tr>
<td>NSW3-R</td>
<td>Up to the base of S2 inclusive</td>
</tr>
<tr>
<td>NSW3-E</td>
<td>Yielding in tensile steel layers up to the base of S3 inclusive</td>
</tr>
</tbody>
</table>

Table B.10: Vibration periods (in seconds) of the elastic lateral modes of the RUAUMOKO and EFfCoS cantilever wall models

<table>
<thead>
<tr>
<th>Wall model</th>
<th>Lateral mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>RUAUMOKO</td>
<td>1.588</td>
</tr>
<tr>
<td>EFfCoS</td>
<td>1.537</td>
</tr>
</tbody>
</table>
### Table B.11: Dynamic analysis results obtained from the EFiCoS model NSW3-E

<table>
<thead>
<tr>
<th>Record</th>
<th>PGA [in g]</th>
<th>$\Delta_{\text{max}}$ [mm]</th>
<th>$V_{b,\text{max}}$ [kN]</th>
<th>Plastic hinges</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LB</td>
<td>UB</td>
<td>LB</td>
<td>UB</td>
</tr>
<tr>
<td>MTL1-M6</td>
<td>0.366</td>
<td>60</td>
<td>77</td>
<td>10790</td>
</tr>
<tr>
<td>MTL2-M6</td>
<td>0.444</td>
<td>82</td>
<td>58</td>
<td>13900</td>
</tr>
<tr>
<td>MTL3-M6</td>
<td>0.399</td>
<td>67</td>
<td>62</td>
<td>10500</td>
</tr>
<tr>
<td>MTL1-M7</td>
<td>0.269</td>
<td>79</td>
<td>81</td>
<td>12820</td>
</tr>
<tr>
<td>MTL2-M7</td>
<td>0.255</td>
<td>75</td>
<td>97</td>
<td>9402</td>
</tr>
<tr>
<td>MTL3-M7</td>
<td>0.308</td>
<td>83</td>
<td>75</td>
<td>10800</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td>74</td>
<td>75</td>
<td>11369</td>
</tr>
</tbody>
</table>

### Table B.12: Dynamic analysis results obtained from the RU-AUMOKO model NSW3-R meshed with one-component beam elements

<table>
<thead>
<tr>
<th>Record</th>
<th>PGA (in g)</th>
<th>$\Delta_{\text{max}}$ [mm]</th>
<th>$V_{b,\text{max}}$ [kN]</th>
<th>Plastic hinges</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LB</td>
<td>UB</td>
<td>LB</td>
<td>UB</td>
</tr>
<tr>
<td>MTL1-M6</td>
<td>0.366</td>
<td>181</td>
<td>138</td>
<td>8480</td>
</tr>
<tr>
<td>MTL2-M6</td>
<td>0.444</td>
<td>122</td>
<td>114</td>
<td>8107</td>
</tr>
<tr>
<td>MTL3-M6</td>
<td>0.399</td>
<td>279</td>
<td>146</td>
<td>7568</td>
</tr>
<tr>
<td>MTL1-M7</td>
<td>0.269</td>
<td>342</td>
<td>133</td>
<td>7126</td>
</tr>
<tr>
<td>MTL2-M7</td>
<td>0.255</td>
<td>158</td>
<td>242</td>
<td>8169</td>
</tr>
<tr>
<td>MTL3-M7</td>
<td>0.308</td>
<td>164</td>
<td>119</td>
<td>7002</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td>208</td>
<td>149</td>
<td>7742</td>
</tr>
</tbody>
</table>
Table B.13: Dynamic analysis results obtained from the RU-AUMOKO model NSW3-R meshed with general two-component beam elements.

<table>
<thead>
<tr>
<th>Record</th>
<th>PGA (in g)</th>
<th>$\Delta_{max}$ [mm]</th>
<th>$V_{h,max}$ [kN]</th>
<th>Plastic hinges</th>
</tr>
</thead>
<tbody>
<tr>
<td>MTL1-M6</td>
<td>0.366</td>
<td>127 106</td>
<td>7957 15490</td>
<td>No No</td>
</tr>
<tr>
<td>MTL2-M6</td>
<td>0.444</td>
<td>149 64</td>
<td>8542 15700</td>
<td>No No</td>
</tr>
<tr>
<td>MTL3-M6</td>
<td>0.399</td>
<td>171 107</td>
<td>9139 12780</td>
<td>No No</td>
</tr>
<tr>
<td>MTL1-M7</td>
<td>0.269</td>
<td>212 91</td>
<td>8981 15180</td>
<td>No No</td>
</tr>
<tr>
<td>MTL2-M7</td>
<td>0.255</td>
<td>110 178</td>
<td>7726 14250</td>
<td>No No</td>
</tr>
<tr>
<td>MTL3-M7</td>
<td>0.308</td>
<td>118 103</td>
<td>9723 12460</td>
<td>No No</td>
</tr>
<tr>
<td>Mean</td>
<td></td>
<td>148 108</td>
<td>8678 14310</td>
<td></td>
</tr>
</tbody>
</table>
Figure B.7: Top wall displacement responses, for the UB case, predicted from the RU-AUMOKO model NSW3-R, meshed with two different types of beam elements, and the EFiCoS model NSW3-E, to ground motions: (a) MTL2-M6, (b) MTL1-M7, (c) MTL2-M7. All responses are for a probable resistance.
Table B.14: Dynamic analysis results obtained from RUAUMOKO and EFiCoS wall models when subjected to the ELCENTRO ground excitation

<table>
<thead>
<tr>
<th>Model</th>
<th>FE Code</th>
<th>Beam Element</th>
<th>$\Delta_{max}$ [mm]</th>
<th>$V_{h,\text{max}}$ [kN]</th>
<th>Plastic hinges</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSW3-E EFiCoS</td>
<td>Multilayers</td>
<td>LB 333 UB 343</td>
<td>LB 24670 UB 16540</td>
<td>Base of S1 and in the vicinity of floor level 8</td>
<td>Base of S1 and in the vicinity of floor level 8</td>
</tr>
<tr>
<td>NSW3-R RUAUMOKO</td>
<td>One-component</td>
<td>LB 421 UB 419</td>
<td>LB 14720 UB 12400</td>
<td>No</td>
<td>Base of S1 and in the vicinity of floor level 8</td>
</tr>
<tr>
<td>NSW3-R RUAUMOKO (general)</td>
<td>Two-component</td>
<td>LB 322 UB 348</td>
<td>LB 18280 UB 15580</td>
<td>Base of S1</td>
<td>Base of S1 and in the vicinity of floor level 8</td>
</tr>
</tbody>
</table>

Table B.15: Vibration periods (in seconds) of the elastic lateral modes of the structural models used for inelastic seismic analysis

<table>
<thead>
<tr>
<th>Structural model</th>
<th>Description</th>
<th>Lateral mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1    2</td>
</tr>
<tr>
<td>NSW-R</td>
<td>Cantilever wall</td>
<td>1.588 0.294</td>
</tr>
<tr>
<td>NSB-R</td>
<td>N-S wall building</td>
<td>1.507 0.290</td>
</tr>
<tr>
<td>EWW-R</td>
<td>Coupled wall</td>
<td>1.555 0.401</td>
</tr>
</tbody>
</table>
Figure B.8: Top wall displacement responses to ELCENTRO ground excitation predicted from the EFICoS wall model NSW3-E and RUAMOKO wall model NSW3-R meshed with: (a) one-component (Giberson) beam elements; (b) general two-component beam elements. All responses are for a probable resistance.
Figure B.9: 2D structural wall models used in RUAUMOKO for inelastic seismic analysis
<table>
<thead>
<tr>
<th>Building component</th>
<th>Type of element</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall</td>
<td>general</td>
</tr>
<tr>
<td></td>
<td>two-component</td>
</tr>
<tr>
<td></td>
<td>beam element</td>
</tr>
<tr>
<td>Columns, slabs &amp;</td>
<td>one-component</td>
</tr>
<tr>
<td>spandrel beams</td>
<td>beam element</td>
</tr>
</tbody>
</table>

For all floor levels:
- 2 x slab section of 0.2 m thick x 3 m wide (column strip)
- Cantilever wall
- Rigid end extension to account for wall width
- Equal lateral displacements between frames at all floor levels

For all floor levels:
- 2 x spandrel beam section SB

Figure B.10: 2D structural model NSB-R of the building (N-S direction) used in RUAUMOKO for inelastic seismic analysis
Appendix C

Earthquake Ground Motion Histories
Figure C.1: Simulated NBCC 2005-compatible ground-motion time histories and associated 5% damped pseudo-absolute acceleration response spectra of the selected M6.0 events at R=30 km tuned for Montréal.
Figure C.2: Simulated NBCC 2005-compatible ground-motion time histories and associated 5% damped pseudo-absolute acceleration response spectra of the selected M7.0 events at R=70 km tuned for Montréal.
Figure C.3: Accelerogram and associated 5% damped pseudo-spectral absolute acceleration response of the Imperial Valley earthquake, May 18, 1940, El Centro site, North-South component (ELCENTRO)
Appendix D

Inelastic Seismic Analysis Results
Table D.1: Maximum curvature ductility ratios ($\mu_\phi$) in the base plastic hinge predicted from the cantilever wall model NSW-R at design lateral drift ($\Delta_d$) and overall drift limit ($\Delta_{\text{limit}}$)

<table>
<thead>
<tr>
<th>Type of resistance</th>
<th>$\Delta_d = 0.20%$</th>
<th>$\Delta_{\text{limit}} = 2.5%$</th>
<th>$(\mu_\phi)_{\text{ult}}^{(1)}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factored</td>
<td>No hinge</td>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>Nominal</td>
<td>No hinge</td>
<td>37</td>
<td>30</td>
</tr>
<tr>
<td>Probable</td>
<td>No hinge</td>
<td>24</td>
<td>20</td>
</tr>
</tbody>
</table>

1- These ultimate curvature ductility ratios correspond to the tensile strain capacity of reinforcing steel, as the concrete compression strain demands are small at ultimate ($< 0.0035$). This strain capacity is taken as 0.05, as suggested by Adebâr et al. (2004) for bounded reinforcing bars embedded in concrete.
Figure D.1: Base shear force versus drift at building roof predicted from (a) cantilever wall model NSW-R, (b) coupled wall model EWW-R
Figure D.2: Predicted damage states and sequences of hinge formation at (a) design lateral drift ($\Delta_d$) (b) overall drift limit ($\Delta_{\text{limit}}$). Predictions are the same for factored, nominal and probable resistances.
Figure D.3: Base moment redistribution predicted from the coupled wall model EWW-R for a probable resistance

Figure D.4: Base shear redistribution predicted from the coupled wall model EWW-R for a probable resistance
### Table D.2: Maximum curvature ductility ratios ($\mu_\phi$) in plastic hinges predicted from the coupled wall model EWW-R at design lateral drift ($\Delta_d = 0.18\%$)

<table>
<thead>
<tr>
<th>Structural member</th>
<th>Floor level</th>
<th>Resistance</th>
<th>Ultimate $\mu_\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupling beam$^{(1)}$</td>
<td>Penthouse</td>
<td>Factored 2.2</td>
<td>Nominal 1.8</td>
</tr>
<tr>
<td></td>
<td>13</td>
<td>Factored 2.3</td>
<td>Nominal 1.9</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>Factored 2.3</td>
<td>Nominal 2.0</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>Factored 2.4</td>
<td>Nominal 2.1</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>Factored 2.4</td>
<td>Nominal 2.1</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>Factored 2.5</td>
<td>Nominal 2.1</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>Factored 2.5</td>
<td>Nominal 2.2</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>Factored 2.5</td>
<td>Nominal 2.2</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>Factored 2.4</td>
<td>Nominal 2.1</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Factored 2.3</td>
<td>Nominal 2.0</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Factored 2.2</td>
<td>Nominal 1.9</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Factored 2.0</td>
<td>Nominal 1.7</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>Factored 1.6</td>
<td>Nominal 1.4</td>
</tr>
<tr>
<td>Tension wall$^{(2)}$</td>
<td>1</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Compression wall$^{(3)}$</td>
<td>1</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

1- The ultimate curvature ductility ratio of 10 corresponds to the inelastic rotational capacity of 0.04 specified in the CSA A23.3-04 for diagonally reinforced coupling beams.

2- The ultimate curvature ductility ratio is for a probable resistance. See note 1 at the bottom of Table D.1.

3- The ultimate curvature ductility ratio is for a probable resistance and corresponds to the concrete compression strain capacity, which is taken as 0.0035.
### Table D.3: Maximum curvature ductility ratios ($\mu_\phi$) in plastic hinges predicted from the coupled wall model EWW-R at overall drift limit ($\Delta_{\text{limit}} = 1.13\%$)

<table>
<thead>
<tr>
<th>Structural member</th>
<th>Floor level</th>
<th>Resistance</th>
<th>Ultimate $\mu_\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coupling beam$^{(1)}$</td>
<td>Penthouse</td>
<td>Factored 15, Nominal 12, Probable 10</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>13</td>
<td>15</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>15</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>15</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>14</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>14</td>
<td>12</td>
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<tr>
<td></td>
<td>8</td>
<td>14</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>14</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>14</td>
<td>12</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>14</td>
<td>12</td>
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<tr>
<td></td>
<td>4</td>
<td>13</td>
<td>11</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>12</td>
<td>11</td>
</tr>
<tr>
<td>Tension wall$^{(2)}$</td>
<td>1</td>
<td>15</td>
<td>16</td>
</tr>
<tr>
<td>Compression wall$^{(3)}$</td>
<td>1</td>
<td>9</td>
<td>8</td>
</tr>
</tbody>
</table>

1. The ultimate curvature ductility ratio of 10 corresponds to the inelastic rotational capacity of 0.04 specified in the CSA A23.3-04 for diagonally reinforced coupling beams.

2. The ultimate curvature ductility ratio is for a probable resistance. See note 1 at the bottom of Table D.1.

3. The ultimate curvature ductility ratio is for a probable resistance and corresponds to the concrete compression strain capacity, which is taken as 0.0035.
Table D.4: Maximum roof drift ($\Delta_{max}$), plastic hinges and associated maximum curvature ductility ratios ($\mu_{\phi}^{max}$) predicted from the cantilever wall model NSW-R under the selected NBCC 2005-compatible ground motions for Montréal.

<table>
<thead>
<tr>
<th>Records</th>
<th>Cantilever wall model NSW-R</th>
<th>1% damping</th>
<th>2% damping</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\Delta_{max}$</td>
<td>Plastic hinges</td>
<td>$\mu_{\phi}^{max}$</td>
</tr>
<tr>
<td>MTL1-M6</td>
<td>0.30%</td>
<td>No</td>
<td>-</td>
</tr>
<tr>
<td>MTL2-M6</td>
<td>0.17%</td>
<td>No</td>
<td>-</td>
</tr>
<tr>
<td>MTL3-M6</td>
<td>0.34%</td>
<td>No</td>
<td>-</td>
</tr>
<tr>
<td>MTL1-M7</td>
<td>0.30%</td>
<td>Base of S1</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Base of S8</td>
<td>7.0</td>
</tr>
<tr>
<td>MTL2-M7</td>
<td>0.41%</td>
<td>Base of S1</td>
<td>1.2</td>
</tr>
<tr>
<td>MTL3-M7</td>
<td>0.36%</td>
<td>No</td>
<td>-</td>
</tr>
<tr>
<td>Mean</td>
<td>0.31%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>COV</td>
<td>0.26</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M+SD</td>
<td>0.39%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum</td>
<td>0.41%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design</td>
<td>0.20%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure D.5: Comparison between design interstorey drifts and interstorey drift response envelopes predicted from the cantilever wall model NSW-R. Predicted envelopes are peak responses to all six selected NBCC 2005-compatible ground motions for Montréal.
APPENDIX D. INELASTIC SEISMIC ANALYSIS RESULTS

(a) 1% damping

(b) 2% damping

Figure D.6: Interstorey drift response envelopes predicted from the cantilever wall model NSW-R
Figure D.7: Comparison between design moment capacity envelope, probable moment resistance and overturning moment response envelopes predicted from the cantilever wall model NSW-R. Predicted envelopes are peak responses to all six selected NBCC 2005-compatible ground motions for Montréal.
Figure D.8: Comparison between design shear forces ($V_p$), factored shear resistance ($V_r$) and shear force response envelopes predicted from the cantilever wall model NSW-R. Predicted envelopes are peak responses to all six selected NBCC 2005-compatible ground motions for Montréal.
Figure D.9: Wall shear force response envelopes predicted from the cantilever wall model NSW-R.
Figure D.10: Comparison between design interstorey drifts and interstorey drift response envelopes predicted from the cantilever wall model NSW-R and the building model NSB-R. Predicted envelopes are peak responses to all six selected NBCC 2005-compatible ground motions for Montréal.
Figure D.11: Comparison between design moment capacity envelope, probable moment resistance and overturning moment response envelopes predicted from the cantilever wall model NSW-R and the building model NSB-R. Predicted envelopes are peak responses to all six selected NBCC 2005-compatible ground motions for Montréal.
Figure D.12: Comparison between design shear forces ($V_p$), factored shear resistance ($V_r$) and shear force response envelopes predicted from the cantilever wall model NSW-R and the building model NSB-R. Predicted envelopes are peak responses to all six selected NBCC 2005-compatible ground motions for Montréal.
Figure D.13: Pseudo-spectral acceleration response spectra

Figure D.14: Interstorey drift predictions obtained from the cantilever wall model NSW-R for 5% damping. Predicted envelopes are peak responses to all six selected NBCC 2005-compatible ground motions for Montréal.
Figure D.15: Wall overturning moment and shear force predictions obtained from the cantilever wall model NSW-R for 5% damping. Predicted envelopes are peak responses to all six selected NBCC 2005-compatible ground motions for Montréal.
**APPENDIX D. INELASTIC SEISMIC ANALYSIS RESULTS**

**Table D.5:** Mean COVs, over building height, of peak interstorey drift (PID), peak wall overturning moment (PWOM), peak wall shear force (PWSF) and peak beam curvature ductility (PBCD) responses to selected NBCC 2005-compatible ground motions for Montréal

<table>
<thead>
<tr>
<th>Model</th>
<th>Description</th>
<th>PID</th>
<th>PWOM(^1)</th>
<th>PWSF</th>
<th>PBCD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1%</td>
<td>2%</td>
<td>1%</td>
<td>2%</td>
</tr>
<tr>
<td>NSW-R</td>
<td>Cantilever wall</td>
<td>0.37</td>
<td>0.37</td>
<td>0.11</td>
<td>0.11</td>
</tr>
<tr>
<td>NSB-R</td>
<td>N-S wall building</td>
<td>0.35</td>
<td>0.39</td>
<td>0.11</td>
<td>0.13</td>
</tr>
<tr>
<td>EWW-R</td>
<td>Coupled wall</td>
<td>0.18</td>
<td>0.20</td>
<td>0.12</td>
<td>0.13</td>
</tr>
</tbody>
</table>

\(^1\) Peak wall bending moment (PWBM) for the coupled wall

**Table D.6:** Maximum roof drift (\(\Delta_{\text{max}}\)), plastic hinges and associated maximum curvature ductility ratios (\(\mu^\phi_{\text{max}}\)) predicted from the building model NSB-R under the selected NBCC 2005-compatible ground motions for Montréal

<table>
<thead>
<tr>
<th>Record</th>
<th>Building model NSB-R</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(\Delta_{\text{max}}) Plastic hinges (\mu^\phi_{\text{max}}) (\Delta_{\text{max}}) Plastic hinges (\mu^\phi_{\text{max}})</td>
</tr>
<tr>
<td>MTL1-M6</td>
<td>0.25% No - 0.21% No -</td>
</tr>
<tr>
<td>MTL2-M6</td>
<td>0.13% No - 0.13% No -</td>
</tr>
<tr>
<td>MTL3-M6</td>
<td>0.26% No - 0.24% No -</td>
</tr>
<tr>
<td>MTL1-M7</td>
<td>0.23% No - 0.23% No -</td>
</tr>
<tr>
<td>MTL2-M7</td>
<td>0.40% Wall: base of S1 1.6 0.37% Wall: base of S1 1.5</td>
</tr>
<tr>
<td>MTL3-M7</td>
<td>0.28% Wall: base of S1 1.1 0.31% Wall: base of S1 1.0</td>
</tr>
<tr>
<td>Mean</td>
<td>0.26% 0.25%</td>
</tr>
<tr>
<td>COV</td>
<td>0.33 0.34</td>
</tr>
<tr>
<td>M+SD</td>
<td>0.34% 0.33%</td>
</tr>
<tr>
<td>Maximum</td>
<td>0.49% 0.37%</td>
</tr>
<tr>
<td>Design</td>
<td>0.20% 0.20%</td>
</tr>
</tbody>
</table>
Figure D.16: Coupling beam curvature ductility ratio envelopes predicted from the coupled wall model EWW-R. Predicted envelopes are peak responses to all six selected NBCC 2005-compatible ground motions for Montréal.
Table D.7: Maximum roof drift ($\Delta_{\text{max}}$), plastic hinges and associated maximum curvature ductility ratios ($\mu_{\phi}^{\text{max}}$) predicted from the coupled wall model EWW-R under the selected NBCC 2005-compatible ground motions for Montréal.

<table>
<thead>
<tr>
<th>Record</th>
<th>1% damping</th>
<th>2% damping</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\Delta_{\text{max}}$</td>
<td>Plastic hinges</td>
</tr>
<tr>
<td>MTL1-M6</td>
<td>0.12%</td>
<td>All beams only</td>
</tr>
<tr>
<td>MTL2-M6</td>
<td>0.10%</td>
<td>All beams only</td>
</tr>
<tr>
<td>MTL3-M6</td>
<td>0.15%</td>
<td>All beams only</td>
</tr>
<tr>
<td>MTL1-M7</td>
<td>0.16%</td>
<td>All beams only</td>
</tr>
<tr>
<td>MTL2-M7</td>
<td>0.17%</td>
<td>All beams only</td>
</tr>
<tr>
<td>MTL3-M7</td>
<td>0.16%</td>
<td>All beams only</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Mean</th>
<th>COV</th>
<th>M+SD</th>
<th>Maximum</th>
<th>Design</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.14%</td>
<td></td>
<td>0.17%</td>
<td>0.17%</td>
<td>0.18%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.18</td>
<td></td>
<td>0.17%</td>
<td>0.20%</td>
</tr>
</tbody>
</table>


Figure D.17: Comparison between design interstorey drifts and interstorey drift response envelopes predicted from the coupled wall model EWW-R. Predicted envelopes are peak responses to all six selected NBCC 2005-compatible ground motions for Montréal.
Figure D.18: Comparison between design moment capacity envelope, probable moment resistance and bending moment response envelopes predicted from the coupled wall model EWW-R. Predicted envelopes are peak responses to all six selected NBCC 2005-compatible ground motions for Montréal.
Figure D.19: C-shaped wall bending moment response envelopes predicted from the coupled wall model EWW-R.
Figure D.20: Comparison between design shear forces \((V_p)\), factored shear resistance \((V_r)\) and shear force response envelopes predicted from the coupled wall model EWW-R. Predicted envelopes are peak responses to all six selected NBCC 2005-compatible ground motions for Montréal.
Figure D.21: C-shaped wall shear force response envelopes predicted from the coupled wall model EWW-R.
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