SEISMIC DESIGN GUIDE FOR MASONRY BUILDINGS

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Canadian Concrete Masonry Producers Association

Canadian Concrete Masonry Producers Association
Region 6 of the National Concrete Masonry Association
CCMPA
Building Strength

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While the authors have tried to be as accurate as possible, they cannot be held responsible for the designs of others that might be based on the material presented in this document. The material included in this document is intended for the use of design professionals who are competent to evaluate the significance and limitations of its contents and recommendations and able to accept responsibility for its application. The authors, and the Canadian Concrete Masonry Producers Association, disclaim any and all responsibility for the applications of the stated principles and for the accuracy of any of the material included in the document.

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The Canadian Concrete Masonry Producers Association (CCMPA) is a non-profit association whose mission is to support and advance the common interests of its members in the manufacture, marketing, research, and application of concrete masonry products and structures. It represents the interests of Region 6 of the National Concrete Masonry Association (NCMA).

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FOREWORD

The Canadian Concrete Masonry Producers Association (CCMPA) is a non-profit association whose mission is to support and advance the common interests of its members in the manufacture, marketing, research, and application of concrete masonry products and structures. The CCMPA represents the producers of concrete masonry products in Canada. Our member firms are engaged in the manufacture of concrete block and concrete brick masonry units used for loadbearing and nonloadbearing applications, and as veneers. The CCMPA represents Canadian interests within the National Concrete Masonry Association, a U.S.-based international association of concrete masonry producers.

The CCMPA supports the educational efforts of Canadian universities and other educational institutions, and the education of the masonry design professional, practitioner and student, both formally and informally. The CCMPA is intimately involved in the development and maintenance of CSA masonry and masonry-related standards. Our standards serve as the basis for manufacturing and specifying concrete masonry materials and products, product and assembly testing, and the structural design and construction of masonry elements. The CCMPA continually develops and disseminates information and design tools needed by designers to deliver state-of-the-art, safe and serviceable, cost-effective masonry elements and structures. As part of this continuing commitment to education, the CCMPA is pleased to sponsor and publish this guide, co-authored by Drs. Anderson and Brzev, two authorities in seismic behaviour and design of masonry.

This is the first edition of the "Seismic Design Guide for Masonry Buildings". This comprehensive and illustrated guide is based on the 2005 edition of the National Building Code of Canada (NBCC) and the 2004 edition of CSA S304.1, "Design of Masonry Structures". Its format and content have been specifically developed to address the needs of the practicing structural engineer designing low and mid-rise masonry buildings and their elements. The Guide describes the behaviour of masonry under seismic loading, explains and rationalizes the basis of the seismic design requirements within the NBCC and S304.1, and provides guidance and assistance to masonry designers on their interpretation and use. It describes and details the appropriate methods of seismic design and analysis, and demonstrates use by illustrative example. The Guide recognizes the high standard of quality control present in modern masonry structures and the advanced methods used in its structural design. There is no similar or comparable guide for the seismic design of masonry in Canada, and no more comprehensive guide internationally.

In particular, the CCMPA acknowledges the commitment to the development of this Guide by its authors and their dedication to masonry education and research. We recognize the past and on-going work by Dr. Anderson, University of British Columbia, who has spearheaded and coordinated the requirements for masonry seismic design in both the 1994 and 2004 editions of CSA S304.1, the Canadian masonry design standard. At the Building Code level, Dr. Anderson also serves as a member of the Canadian National Committee for Earthquake Engineering (CANCEE). CANCEE is the Standing Committee on Earthquake Design of NBCC Part 4. Dr. Anderson's liaison between CANCEE and S304.1 has been eminently important for developing seismic requirements in the S304.1 standard and harmonizing these requirements with those of the NBCC. Dr. Brzev, British Columbia Institute of Technology, brings to us, her vast international experience and understanding in behaviour and design of concrete and masonry elements and structures, and earthquake engineering. She is the author of numerous research publications and a co-author of “Reinforced Concrete Design, A Practical Approach”. The Canadian Concrete Masonry Producers Association is very grateful for the support offered by both of these educators and researchers.

This Guide was developed on the basis of the Limit States Design method of CSA Standard S304.1-04. The references to this standard in this Guide neither duplicate nor replace this standard. Therefore, it is recommended that the user of this Guide obtain a copy of CSA S304.1-04, “Masonry Design for Structures” developed and published by the Canadian Standards Association (www.csa.ca).

The timely appearance of this guide will give rise to a new generation of masonry buildings and their proliferation.

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PREFACE

This document is intended to assist practicing structural engineers in designing masonry buildings for seismic load effects according to the National Building Code of Canada 2005 (NBCC 2005) and the CSA S304.1-04 masonry design standard. The document provides a commentary which explains the underlying theoretical background and rationale for the CSA S304.1-04 seismic design provisions. Changes in the seismic design provisions contained in Part 4 of the NBCC 2005 and CSA S304.1-04, and their impact on masonry design and construction are discussed.

This document is a comprehensive state-of-the-art guide on seismic design and construction of masonry structural elements for low to mid-rise structures, such as warehouses, industrial buildings, schools, commercial buildings, and residential/hotel structures. It is restricted to masonry structures designed and constructed using concrete block units. Consideration of the slenderness effects in tall masonry walls is beyond the scope of this document.

The material is presented in a simple and user-friendly manner. It facilitates the application of seismic design provisions and cross-referencing of code clauses for designers. The document has been developed in a modular form, with the content divided into four chapters, each of which can be used in a stand-alone manner. The appendices contain useful resources such as design procedures and research background for some of the design provisions. For easy reference, relevant code clauses are identified by framed boxes wherever appropriate.

Chapter 1 provides a review of the general seismic design provisions contained in Part 4 of NBCC 2005, including seismic hazard levels, and the equivalent static force procedure. It discusses key design parameters such as irregularities, torsion, height limitations, and the ductility and overstrength factors for masonry structures. Additionally, there is an introduction to the dynamic analysis of structures to assist in understanding pertinent code provisions. Since there are major changes to the seismic provisions in NBCC 2005, some comparisons with the previous 1995 edition of the building code are included in Appendix A.

Chapter 2 provides an overview of seismic design requirements for reinforced masonry walls. Relevant CSA S304.1-04 design requirements are presented along with related commentary that provides detailed explanations of the code provisions. Topics include reinforced masonry shear walls subjected to in-plane and out-of-plane seismic loads, and a detailed discussion of the CSA S304.1 seismic design requirements. A few special topics such as masonry infill walls, stack pattern walls, masonry veneers, and construction-related issues are also included. Changes in CSA S304.1-04 seismic design requirements from the previous (1994) edition are identified and discussed, along with their design implications. Appendix B contains resources related to the Chapter 2 content, including findings of research studies and foreign code provisions related to seismic design of masonry structures.

Chapter 3 summarizes differences in seismic design provisions contained in the 1995 and 2005 editions of NBCC, and the 1994 and 2004 editions of CSA S304.1. Designers who are already familiar with the detailed seismic and masonry design issues discussed in Chapters 1 and 2 may wish to move directly to this summary chapter. The differences in code provisions are presented in a tabular format. This chapter also summarizes the results of a case study of a typical warehouse building designed to both NBCC 1995 and NBCC 2005. The study considers three Canadian locations characterized by different seismic hazards (Vancouver, Calgary, and Toronto).

Chapter 4 provides illustrative design examples of seismic load calculations and distribution of forces to members according to NBCC 2005, and the design of loadbearing and nonloadbearing masonry elements to CSA S304.1-04. The layout of masonry buildings and the mechanical properties of their components in the examples are chosen to reflect situations often encountered in design practice, particularly as they relate to torsionally unsymmetric buildings. These examples are laid out in a step-by-step manner, with ample explanations and appropriate illustrations provided to clarify the design process. Appendix C provides relevant background information for the design examples, including an extensive discussion of in-plane wall stiffness. Appendix D contains design aids used in the Chapter 4 examples.

A list of key references, useful for supplementary reading for those interested in pursuing the subject further, is also included.

ACKNOWLEDGMENTS

It would not be possible to develop and finalize a document of this size without support and assistance provided by several individuals and organizations. The authors are grateful to Canadian Concrete Masonry Producers Association (CCMPA) for giving them an opportunity to undertake this project. The authors are indebted to Gary Sturgeon, P.Eng., Director of Technical Services, CCMPA for spearheading the development of this document and for the guidance and comments provided during its development. The authors gratefully acknowledge Bill McEwen,P.Eng., Executive Director of the Masonry Institute of BC, for providing valuable review comments, guidance and encouragement during the development of this document. Bill shared some of his practical field insights in the Constructability Issues section of the guide.

The authors gratefully acknowledge the following colleagues from Vancouver, BC who have reviewed the manuscript and made valuable comments: Diether Jacobsohn,B.Eng. of WSB Consulting Structural Engineers, Richard Mastschuch, P.Eng. of Pomeroy Consulting Engineers Ltd., Rob Hall, P.Eng. of Bush, Bohlman & Partners, Primo Cajiao,P.Eng. of Sandwell Engineering Inc., and Andy Vizer, P.Eng. of the Cement Association of Canada. The authors would also like to acknowledge the following reviewers retained by the CCMPA: Dr. Ted Sherwood and Dr. Jag Humar, Department of Civil and Environmental Engineering, Carleton University; Dave Laird, P.Eng. and Greg Andrews, P.Eng. of Halsall Associates Ltd., Toronto, ON; and Dr. Mark Hagel of CCMPA. The authors are grateful to Doug Birch, P.Eng.,Struct.Eng. of Krahn Engineering for performing technical review of the design examples included in the guide.

Draft version of the document was used in the fall of 2008 as a teaching resource for the course E1 Masonry Design of Buildings, which is offered in Vancouver, B.C. as a part of the Certificate of Structural Engineering Program sponsored by the Structural Engineers Association of BC and the Department of Civil Engineering, UBC. The authors appreciate the feedback and suggestions made by the students enrolled in the course, and would like to acknowledge in particular Joseph Tam, P.Eng., of AMEC, Vancouver, who has done a thorough review of the document.

Last, but not the least, the authors would like to thank Natalia Leposavic, M.Arch. of BCIT, Vancouver, for preparing excellent drawings included in this document.

CREDITS

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1 Seismic Design Provisions of the National Building Code of Canada 2005

1.1 Introduction

This chapter provides a review of the seismic design provisions in the 2005 National Building Code of Canada (NBCC 2005). Additionally, there is an introduction to the dynamic analysis of structures to assist in understanding the NBCC provisions. Since there are major changes to the seismic provisions reflected in NBCC 2005, some comparisons will be made to the previous edition of the building code, NBCC 1995, and this is covered in more detail in Appendix A.

In the past, building structures in many areas of Canada did not have to be designed for earthquakes. However, after the NBCC 2005 was issued and adopted by the Provinces, structures in some additional areas must now be designed for earthquakes, especially if the structure is an important or post-disaster building, or if it is located on a soft soil site. Since many engineers in these regions have not had experience in seismic design and now may have to include such design in their practice, this guideline has been prepared to explain the seismic provisions included in the NBCC 2005 and CSA S304.1-04, and to point out the recent changes in these two documents as they pertain to masonry design.

1.2 Background

Seismic design of masonry structures became an issue following the 1933 Long Beach, California earthquake in which school buildings suffered damage that would have been fatal to students had the earthquake occurred during school hours. At that time, a seismic lateral load equal to the product of a seismic coefficient and the structure weight had to be considered in those areas of California known to be seismically active. Strong motion instruments that could measure the peak ground acceleration or displacement were developed around that time, and in fact, the first strong motion accelerogram was recorded during the 1933 Long Beach earthquake. However, in this era the most widely used strong ground motion acceleration record was measured at El Centro during the 1940 Imperial Valley earthquake in southern California. The 1940 El Centro record became famous and is still used by many researchers studying the effect of earthquakes on structures.

With the availability of ground motion acceleration records (also known as acceleration time history records), it was possible to determine the response of simple structures modelled as single degree of freedom systems. After computers became available in the 1960s it was possible to develop more complex models for analyzing the response of larger structures. The advent of computers has also had a huge impact on the ability to predict the ground motion hazard at a site, and in particular, on probabilistic predictions of hazard on which the NBCC seismic hazard model is based.

1.3 Design and Performance Objectives

For many years, seismic design philosophy has been founded on the understanding that it would be too expensive to design most structures to remain elastic under the forces that the earthquake ground motion creates. Accordingly, most modern building codes allow structures to be designed for forces lower than the elastic forces with the result that such structures may be damaged in an earthquake, but they should not collapse, and the occupants should be able to safely evacuate the building. The past and present NBCC editions follow this philosophy and allow for lateral design forces smaller than the elastic forces, but impose detailing requirements so that the inelastic response remains ductile and a brittle failure is prevented.

Research studies have shown that for most structures, the lateral displacements or drifts are about the same irrespective of whether the structure remains elastic or it is allowed to yield and experience inelastic (plastic) deformations. This is known as the equal displacement rule and will be discussed later in this chapter, as it forms the basis for many of the code provisions.

The seismic response of a building structure depends on several factors, such as the structural system and its dynamic characteristics, the building materials and design details, but probably the most important is the expected earthquake ground motion at the site. The expected ground motion, termed the *seismic hazard*, can be estimated using probabilistic methods, or be based on deterministic means if there is an adequate history of large earthquakes on identifiable faults in the immediate vicinity of the site.

Canada generally uses a probabilistic method to assess the seismic hazard, and over the years, the probability has been decreasing, from roughly a 40% chance (probability) of being exceeded in 50 years in the 1970s (corresponding to 1/100 per annum probability, also termed the 100 year earthquake), to a 10% in 50 year probability in the 1980s (the 475 year earthquake), to finally a 2% in 50 year probability (the 2475 year earthquake) used for NBCC 2005. The latest change was made so that the risk of building failure in eastern and western Canada would be roughly the same (Adams and Atkinson, 2003), as well as to recognize that an acceptable probability of severe building damage in North America from seismic activity is about 2% in 50 years. Despite the large changes over the years in the probability level for the seismic hazard determination, the seismic design forces have not changed appreciably because other factors in the NBCC design equations have changed to compensate for these higher hazard values. Thus, while the code seismic design *hazard* has been rising over the years, the seismic *risk* of failure of buildings designed according to the code has not changed greatly.

A comparison of building designs performed according to the NBCC 1995 and the NBCC 2005 will show an increase in design level forces in some areas of Canada and a decreased level in other areas, however it is expected that the overall difference between these designs is not significant (see Appendix A for more details).

The NBCC 2005 has taken a more rational approach towards seismic design than have previous editions, in that the seismic hazard has been assessed for a certain probability related to risk of severe building damage, with the building designed with no empirical or calibrating factors. The real strength of the building has been utilized in the design, so that at this level of ground motion it should not collapse but could be severely damaged. Thus, the probability of severe damage or near collapse is about 1/2475 per annum, or about 2% in the predicted 50-year life span of the structure. When compared to wind or snow loads, which are based on the 1 in 50 year probability of not being exceeded, the 1 in 2475 year probability for seismic design appears inconsistent. However, unlike design for seismic loads, design for wind and snow loads uses load and material performance factors, and so the resulting probability of failure is expected to be smaller than that for earthquakes. Seismic design does use material resistance factors, ϕ factors, in assessing member capacity, but they are effectively cancelled out by the overstrength factor, R_o (which will be described later), used to reduce the seismic forces.

Work on new model codes around the world is leading to what is described as, "Performance Based Design", a concept that is already being applied by some designers working with owners who have concerns that building damage will have an adverse effect on their ability to maintain their business. NBCC 2005 only addresses one performance level, that of collapse prevention and life safety, and is essentially mute on serviceability during smaller seismic events that are expected to occur more frequently. Performance based design attempts to minimize the cost of earthquake losses by weighing the cost of repair, and cost of lost business, against an increased cost of construction.

1.4 Response of Structures to Earthquakes

1.4.1 Elastic Response

When an earthquake strikes, the base of a building is subject to lateral motion while the upper part of the structure initially is at rest. The forces created in the structure from the relative displacement between the base and upper portion cause the upper portion to accelerate and displace. At each floor the lateral force required to accelerate the floor mass is provided by the forces in the vertical members. The floor forces are inertial forces, not externally applied forces such as wind loads, and exist only as long as there is movement in the structure.

Earthquakes cause the ground to shake for a relatively short time, 15 to 30 seconds of strong ground shaking, although movements may go on for a few minutes. The motion is cyclic and the response of the structure can only be determined by considering the dynamics of the problem. A few important dynamic concepts are discussed below.

Consider a simple single-storey building with masonry walls and a flat roof. The building can be represented by a Single Degree of Freedom (SDOF) model (also known as a stick model) as shown in Figure 1-1a. The mass, M, lumped at the top, represents the mass of the roof and a fraction of the total wall mass, while the column represents the combined wall stiffness, K, in the direction of earthquake ground motion. If an earthquake causes a lateral deflection, Δ, at the top of the building, Figure 1-1b, and if the building response is elastic with stiffness, K, then the lateral inertial force, F, acting on the mass M will be

$$F = K \cdot \Delta$$

When the mass of a SDOF un-damped structure is allowed to oscillate freely, the time for a structure to complete one full cycle of oscillation is called the period, T, which for the SDOF system shown is given by

$$T = 2\pi\sqrt{\frac{M}{K}} \quad \text{(seconds)}$$

Instead of period, the term *natural frequency*, ω, is often used in seismic design. It is related to the period as follows

$$\omega = \frac{2\pi}{T} = \sqrt{\frac{K}{M}} \quad \text{(radians/sec)}$$

Frequency is sometimes also expressed in Hertz, or cycles per second, instead of radians/sec, denoted by the symbol ω_{cps}, where

$$\omega_{cps} = \frac{1}{T} = \frac{\omega}{2\pi}$$

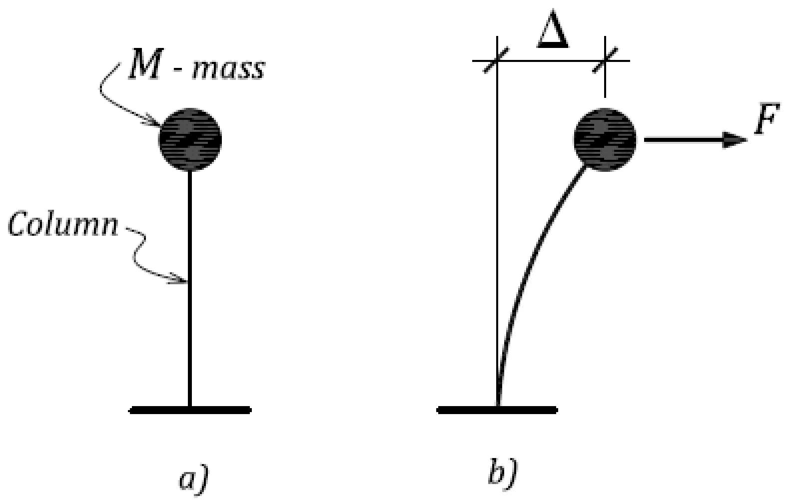


Figure 1-1. SDOF system: a) stick model; b) displaced position.

As the structure vibrates, there is always some energy loss which will cause a decrease in the amplitude of the motion over time - this phenomenon is called *damping*. The extent of damping in a building depends on the materials of construction, its structural system and detailing, and the presence of architectural components such as partitions, ceilings and exterior walls. Damping is usually modelled as viscous damping in elastic structures, and hysteretic damping in structures that demonstrate inelastic response. In seismic design of buildings, damping is usually expressed in terms of a *damping ratio,* β, which is described in terms of a percentage of critical viscous damping. Critical viscous damping is defined as the level of damping which brings a displaced system to rest in a minimum time without oscillation. Damping less than critical, an under-damped system, allows the system to oscillate; while an over-damped system will not oscillate but take longer than the critically damped system to come to rest. Damping has an influence on the period of vibration, *T*, however this influence is minimal for lightly damped systems, and in most cases is ignored for structural systems. For building applications, the damping ratio can be as low as 2%, although 5% is used in most seismic calculations. Damping in a structure increases with displacement amplitude since with increasing displacement more elements may crack or become slightly nonlinear. For linear seismic analysis viscous damping is usually taken as 5% of critical as the structural response to earthquakes is usually close to or greater than the yield displacement. A smaller value of viscous damping is usually used in non-linear analyses as hysteretic damping is also considered.

One of the most useful seismic design concepts is that of the *response spectrum*. When a structure, say the SDOF model shown in Figure 1-1, is subjected to an earthquake ground motion, it cycles back and forth. At some point in time the displacement relative to the ground and the absolute acceleration of the mass reach a maximum, Δ_{max} and a_{max}, respectively. Figure 1-2a shows the maximum displacement plotted against the period, T. Denote the period of this structure as T_1. If the dynamic properties, i.e. either the mass or stiffness change, the period will change, say to T_2. As a result, the maximum displacement will change when the structure is subjected to the same earthquake ground motion, as indicated in Figure 1-2b. Repeating the above process for many different period values and then connecting the points produces a plot like that shown in Figure 1-2c, which is termed the *displacement response spectrum.* The spectrum so determined corresponds to a specific input earthquake motion and a specific damping ratio, β. The same type of plot could be constructed for the maximum acceleration, $a_{\max}$, rather than the displacement, and would be termed the *acceleration response spectrum.*

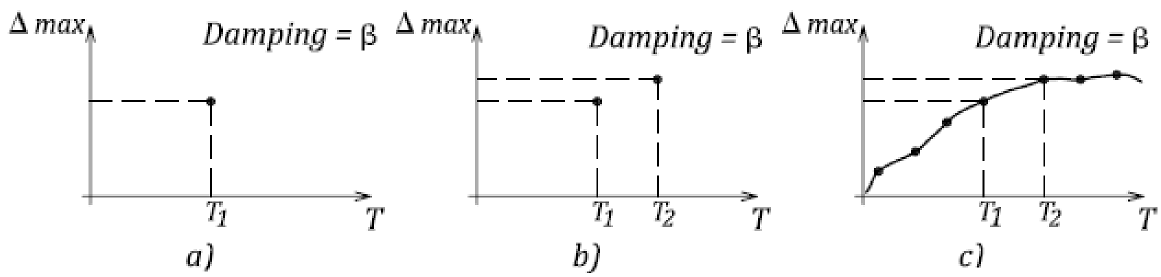


Figure 1-2. Development of a displacement response spectrum - maximum displacement response for different periods T : a) $T = T_1$; b) $T = T_2$; c) many values of T.

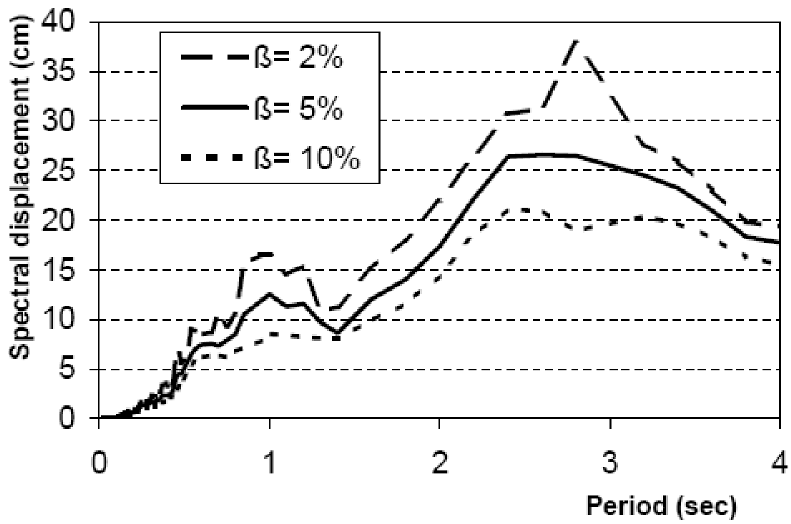
Figure 1-3a shows the displacement response spectrum for the 1940 El Centro earthquake at different damping levels. Note that the displacements decrease with an increase in the damping ratio, β, from 2% to 10%. Figure 1-3b shows the acceleration response spectrum for the same earthquake. For the small amount of damping present in the structures, the maximum acceleration, $a_{\max}$, occurs at about the same time as the maximum displacement, $\Delta_{\max}$, and these two parameters can be related as follows

$$a_{\max} = \left(\frac{2\pi}{T}\right)^2 \Delta_{\max}$$

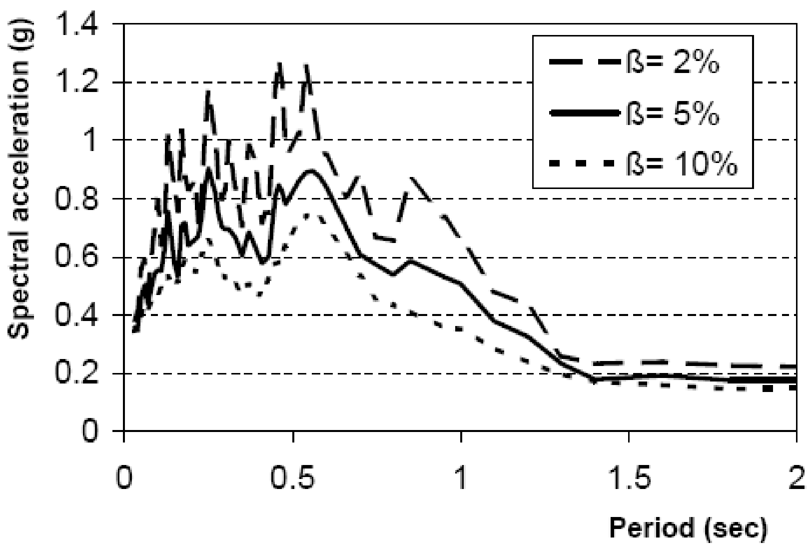
Thus, by knowing the spectral acceleration, it is possible to calculate the displacement spectral values and vice versa. It is also possible to generate a response spectrum for maximum velocity. Except for very short and very long periods, the velocity, $v_{\max}$, is closely approximated by

$$v_{\max} = \left(\frac{2\pi}{T}\right) \Delta_{\max}$$

This is generally called the pseudo velocity response spectrum as it is not the true velocity response spectrum.



a)



b)

Figure 1-3. Response spectra for the 1940 El Centro NS earthquake at different damping levels: a) displacement response spectrum; b) acceleration response spectrum.

The response spectrum can be used to determine the maximum response of a SDOF structure, given its fundamental period and damping, to a specific earthquake acceleration record. Different earthquakes produce widely different spectra and so it has been the practice to choose several earthquakes (usually scaled) and use the resulting average response spectrum as the *design spectrum*. For years, the NBCC seismic provisions have used this procedure where the design spectrum for a site was described by one or two parameters, either peak ground acceleration and/or peak ground velocity, that were determined using probabilistic means.

More recently, probabilistic methods have been used to determine the spectral values at a site for different structural periods. Figure 1-4 shows the 5% damped acceleration response spectrum for Vancouver used in developing the NBCC 2005. This is a uniform hazard response spectrum, i.e., spectral accelerations corresponding to different periods are based on the same probability of being exceeded, that is, 2% in 50 years. This will be discussed further in Section 1.5.1.

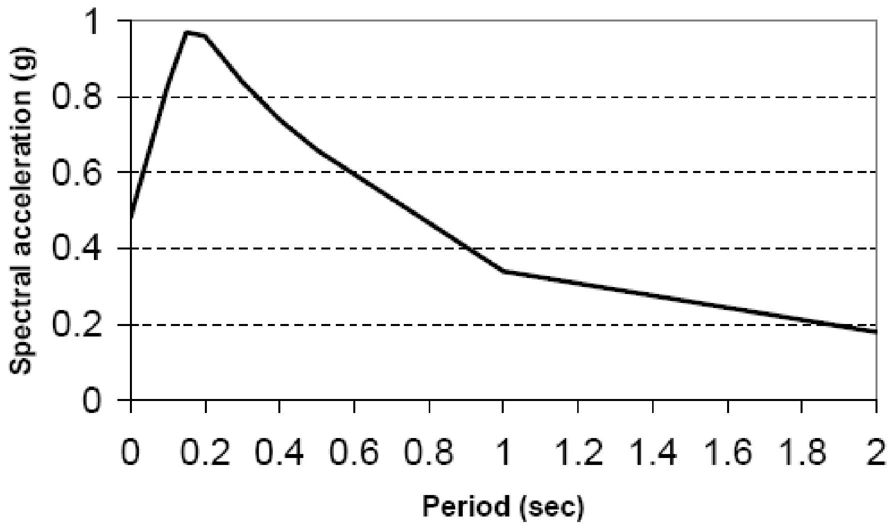


Figure 1-4. Uniform hazard acceleration response spectrum for Vancouver, 2% in 50 year probability, 5% damping.

1.4.2 Inelastic Response

For any given earthquake ground motion and SDOF elastic system it is possible to determine the maximum acceleration and the related inertia force, F_{el} , and the maximum displacement, Δ_{el} . Figure 1-5a shows a force-displacement relationship with the maximum elastic force and displacement indicated. If the structure does not have sufficient strength to resist the elastic force, F_{el} , then it will yield at some lower level of inertia force, say at lateral force level, F_y . It has been observed in many studies that a structure with a nonlinear cyclic force-displacement response similar to that shown in Figure 1-5b will have a maximum displacement that is not much different from the maximum elastic displacement. This is indicated in Figure 1-5c where the inelastic (plastic) displacement, Δ_u , is shown just slightly greater than the elastic displacement, Δ_{el} . This observation has led to the *equal displacement rule*, an empirical rule which states that the maximum displacement that the structure reaches in an earthquake is independent of its yield strength, i.e. irrespective of whether it demonstrates elastic or inelastic response. The equal displacement rule is thought to hold because the nonlinear response softens the structure and so the period increases, thereby giving rise to increased displacements. However, at the same time, the yielding material dissipates energy that effectively increases the damping (the energy dissipation is proportional to the area enclosed by the force-displacement loops, termed hysteresis loops). Increased damping tends to decrease the displacements; therefore, it is possible that the two effects balance one another with the result that the elastic and inelastic displacements are not significantly different.

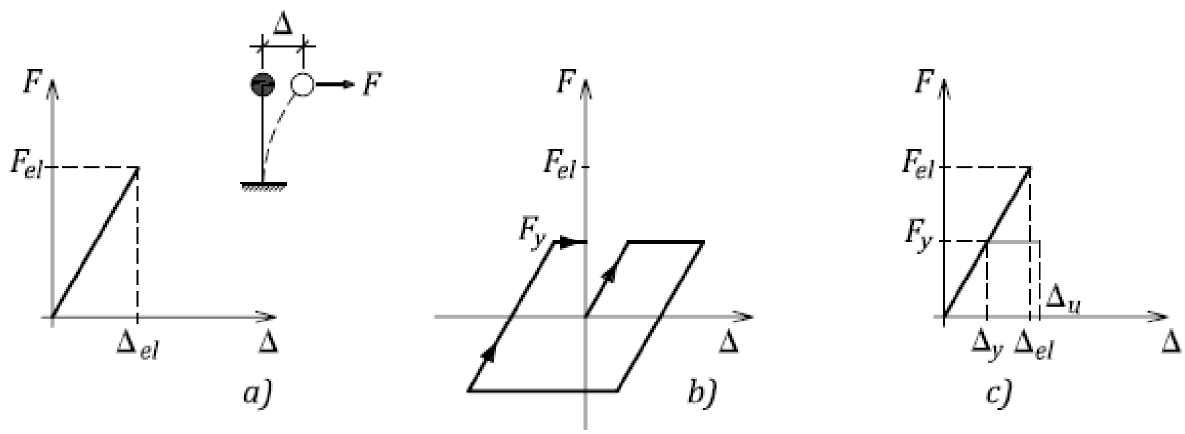


Figure 1-5. Force-displacement relationship: a) elastic response; b) nonlinear (inelastic) response; c) equal displacement rule.

There are limits beyond which the equal displacement rule does not hold. In short period structures, the nonlinear displacements are greater than the elastic displacements, and for very long period structures, the maximum displacement is equal to the ground displacement. However, the equal displacement rule is, in many ways, the basis for the seismic provisions in many building codes which allow the structure to be designed for forces less than the elastic forces. But there is always a trade-off, and the lower the yield strength, the larger the nonlinear or inelastic deformations. This can be inferred from Figure 1-5c where it is noted that the difference between the nonlinear displacement, Δ_u , and yield displacement, Δ_y , which represents the inelastic deformation, would increase as the yield strength decreases. Inelastic deformations generally relate to increased damage, and the designer needs to ensure that the strength does not deteriorate too rapidly with subsequent loading cycles, and that a brittle failure is prevented. This can be achieved by additional "seismic" detailing of the structural members, which is usually prescribed by the material standards. For example, in reinforced concrete structures, seismic detailing consists of additional confinement reinforcement that ensures ductile performance at critical locations in beams, columns, and shear walls. In reinforced masonry structures, it is difficult to provide similar confinement detailing, and so restrictions are placed on limiting the reinforcement spacing, on levels of grouting, and on certain strain limits in the masonry structural components (e.g. shear walls) which provide resistance to seismic loads (see Chapter 2 for more details on seismic design of masonry shear walls).

1.4.3 Ductility

Ductility relates to the capacity of the structure to undergo inelastic displacements. For the SDOF structure, whose force-displacement relation is shown in Figure 1-5c the displacement ductility ratio, μ_Δ , is a measure of damage that the structure might undergo and can be expressed as

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y}$$

The ratio between the maximum elastic force, F_{el} , and the yield force, F_y , is given by the force reduction factor, R , defined as

$$R = \frac{F_{el}}{F_y}$$

If the material is elastic-perfectly plastic, i.e. there is no strain hardening as it yields (see Figure 1-5b), and if Δ_u is equal to Δ_{el}, then it can be shown that μ_Δ is equal to R.

For different types of structures and detailing requirements, most building codes tend to prescribe the R value while not making reference to the displacement ductility ratio, μ_Δ, thus implying that the μ_Δ and R values would be similar.

1.4.4 A Primer on Modal Dynamic Analysis Procedure

The main objective of this section is to explain how more complex multi-degree-of-freedom structures respond to earthquake ground motions and how such response can be quantified in a form useful for structural design. This background should be helpful in understanding the NBCC seismic provisions.

1.4.4.1 Multi-degree-of-freedom systems

The idea of modelling the building as a SDOF structure was introduced in Section 1.4.1, and the dynamic response to earthquake ground motions was developed in terms of a response spectrum. Such a simple model might well represent the lateral response of a single storey warehouse building with flexible walls or bracing system, and with a rigid roof system where the roof comprises most of the weight (mass) of the structure. However, this is not a good model for a masonry warehouse with a metal deck roof, where the walls are quite stiff and the deck is flexible and light relative to the walls. Such a system requires a more complex model using a multi-degree-of-freedom (MDOF) system. A shear wall in a multi-storey building is another example of a MDOF system.

Figure 1-6 shows two examples of MDOF structures. A simple four-storey structure is shown in Figure 1-6a, and a simple MDOF model for this building consists of a column representing the stiffness of vertical members (shear walls or frames), with the masses lumped at the floor levels. If the floors are rigid, it can be assumed that the lateral displacements at every point in a floor are equal, and the structure can be modelled with one degree of freedom (DOF) at each floor level (a DOF can be defined as lateral displacement in the direction in which the structure is being analyzed). This will result in as many degrees of freedom as number of floors, so this building can be modelled as a 4-DOF system. It must also be assumed that there are no torsional effects, that is, there is no rotation of the floors about a vertical axis (torsional effects will be discussed later in Section 1.5.9). The analysis will be the same irrespective of the lateral force resisting system (a shear wall or a frame), aside from details in finding the lateral stiffness matrix for the floor displacements.

The warehouse building shown in Figure 1-6b is another example of a MDOF structure. The walls are treated as a single column with some portion of the wall and roof mass, M_1, located at the top. The roof can be treated as a spring (or several springs) with the remaining roof mass, M_2, attached to the spring(s). How much mass to attach to each degree of freedom, and how to determine the stiffness of the roof, are major challenges in this case.

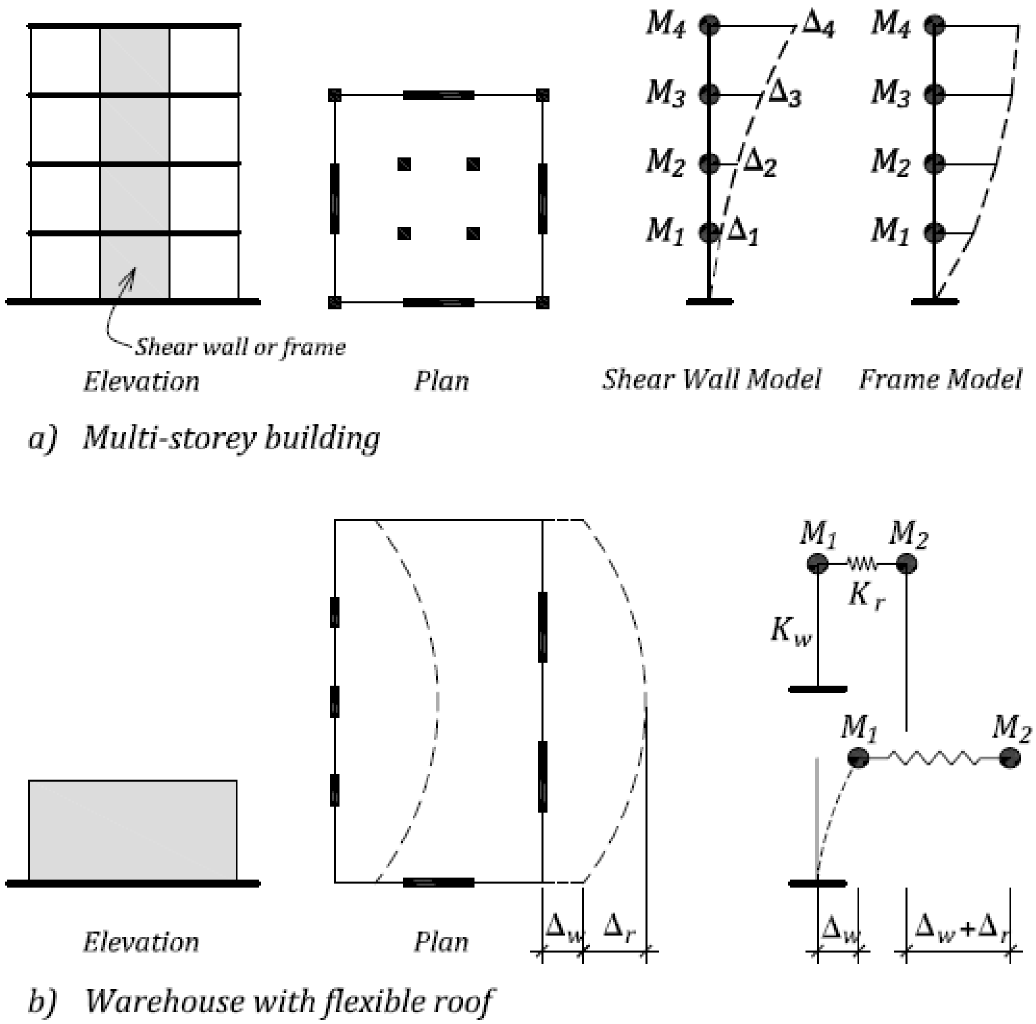


Figure 1-6. MDOF systems: a) multi-storey shear wall building; b) warehouse with flexible roof.

1.4.4.2 Seismic analysis methods
The question of interest to structural engineers is how to determine a realistic seismic response for MDOF systems? The possible approaches are:

- static analysis, and
- dynamic analysis (modal analysis or time history method).

The simplest method is the *equivalent static analysis procedure* (also known as the quasi-static method) in which a set of static horizontal forces is applied to the structure (similar to a wind load). These forces are meant to emulate the maximum effects in a structure that a dynamic analysis would predict. This procedure works well when applied to small, simple structures, and also to larger structures if they are regular in their layout.

NBCC 2005 specifies a dynamic analysis as the default method. The simplest type of dynamic analysis is the *modal analysis method*. This method is restricted to linear systems, and consists of a dynamic analysis to determine the mode shapes and periods of the structure, and then uses a response spectrum to determine the response in each mode. The response of each mode is independent of the other modes, and the modal responses can then be combined to determine the total structural response. In the next section, the modal analysis procedure will be explained with an example.

The second type of dynamic analysis is the *time history method.* This consists of a dynamic analysis model subjected to a time-history record of an earthquake ground motion. Time history analysis is a powerful tool for analyzing complex structures and can take into account nonlinear structural response. This procedure is complex and time-consuming to perform, and as such, not warranted for low-rise and regular structures. It requires an advanced level of knowledge of the dynamics of structures and it is beyond the scope of this document. For detailed background on dynamic analysis methods the reader is referred to Chopra (2007).

1.4.4.3 Modal analysis procedure: an example

Consider a four-storey shear wall building example such as that shown in Figure 1-6a. The building can be modelled as a stick model, with a weight, W , of 2,000 kN lumped at each floor level, and a uniform floor height of 3 m (see Figure 1-7). For simplicity, the wall stiffness and the masses are assumed uniform over the height. This model is a MDOF system with four degrees of freedom consisting of a lateral displacement at each storey level. A MDOF system has as many modes of vibration as degrees of freedom. Each mode has its own characteristic shape and period of vibration. The periods are given in Table 1-1, the four mode shapes are given in Table 1-2 and shown in Figure 1-7. In this example, the stiffness has been adjusted to give a first mode period of 0.4 seconds, which is representative of a four-storey structure based on a simple rule-of-thumb that the fundamental period is on the order of 0.1 sec per floor. Note that the first mode, also known as the *fundamental mode*, has the longest period. The first mode is by far the most important for determining lateral displacements and interstorey drifts, but higher modes can substantially contribute to the forces in structures with longer periods. In this example the mode shapes have been normalized so that the largest modal amplitude is unity.

For linear elastic structures, the equations governing the response of each mode are independent of the others provided that the damping is prescribed in a particular manner. Thus, the response in each mode can be treated in a manner similar to a SDOF system, and this allows the maximum displacement, moment and shear to be calculated for each mode. In the final picture, the modal responses have to somehow be combined to find the design forces (this will be discussed later in this section). Modal analysis can be performed by hand calculation for a simple structure, however, in most cases, the use of a dynamic analysis computer program would be required.

Knowing the mode shapes and the mass at each level, it is possible to calculate the *modal mass* for each mode, which is given in Table 1-1 as a fraction of the total mass of the structure. The modal masses are representative of how the mass is distributed to each mode, and the sum of the modal masses must add up to the total mass. When doing modal analysis, a sufficient number of modes should be considered so that the sum of the modal masses adds up to at least 90% of the total mass. In the example here this would indicate that only the first two modes would need to be considered (0.696 + 0.210 = 0.906).

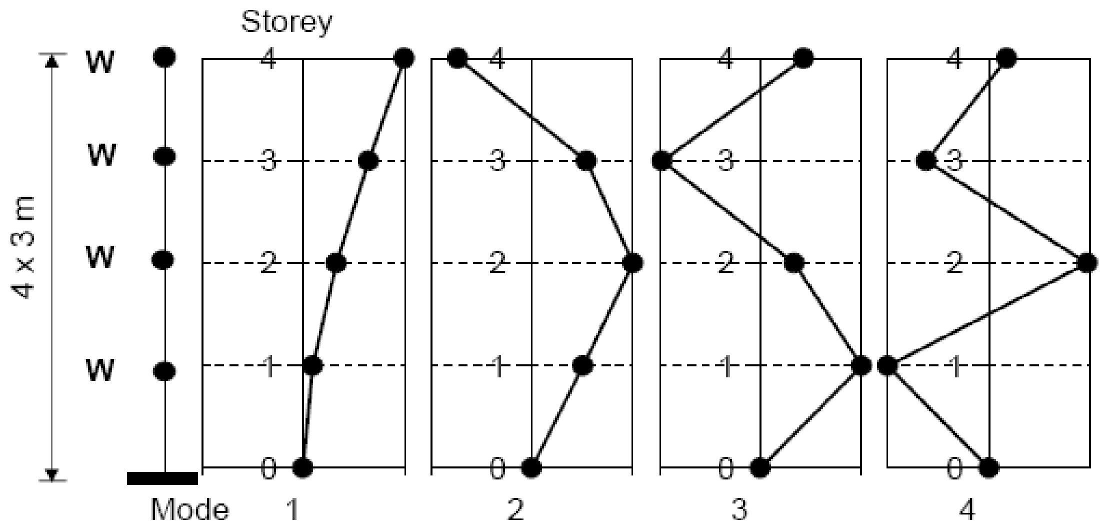


Figure 1-7. Four-storey shear wall building model and modal shapes.

As an example of how the different modes can be used to determine the structural response, Figure 1-8 shows a typical design acceleration response spectrum which will be used to determine the modal displacements and accelerations. The four modal periods are indicated on the spectrum (note that only the first two periods are identified on the diagram; T_1=0.40 and T_2=0.062 sec) and the spectral acceleration S_a at each of the periods is given in Table 1-3.

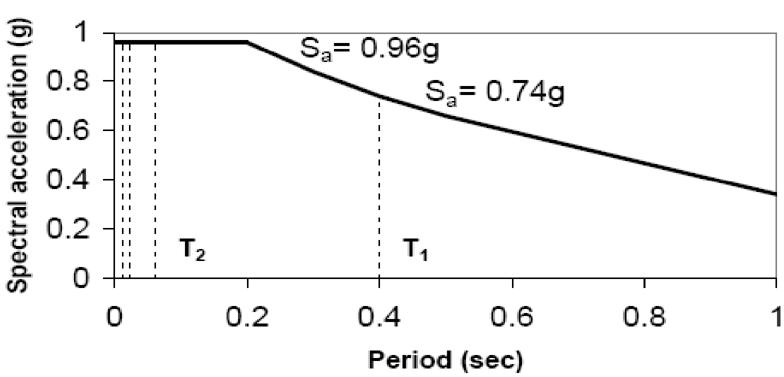


Figure 1-8. Design acceleration response spectrum.

A very useful feature of the modal analysis procedure gives the base shear in each mode as a product of the modal mass and the spectral acceleration S_a for that mode, as shown in Table 1-3. For example, the base shear for the first mode is equal to (8000kN x 0.696) x 0.74 = 4127 kN). Note that the spectral acceleration is higher for the higher modes, but because the modal mass for these modes is smaller, the base shear is smaller. The inertia forces from each floor mass act in the same directions as the mode shape, that is, some forces are positive while others are negative for the higher modes (refer to mode shapes shown in Figure 1-7). It can be seen from the figure that the forces from the first mode all act in the same direction at the same time, while the higher modes will have both positive and negative forces. Thus the base shear from the first mode is usually larger than that from the other modes.

The modal base shears shown in Table 1-3 are the maximum base shears for each mode. It is very unlikely that these forces will occur at the same time during the ground shaking, and they could have either positive or negative signs. Summing the contribution of each mode where all values are taken as positive, known as the absolute sum (ABS) method, produces a very high upper bound estimate of the total base shear. Statistical analyses have shown that the square-root-of-the-sum-of-the squares (RSS) procedure, whereby the contribution of each mode is squared, and the square root is then taken of the sum of the squares, gives a reasonably good estimate of the modal sum, especially if the modal periods are widely separated.

Table 1-3 shows the base shear values estimated by the two methods and gives an indication of the conservatism of the ABS method for this case (total base shear of 6,462 kN), where the modal periods are widely separated, and use of the RSS method is appropriate since it gives a lower total base shear value of 4,468 kN. Note that there is a third method that is incorporated in many modal analysis programs called the complete-quadratic-combination (CQC) method. This method should be used if the periods of some of the modes being combined are close together, as would be the case in many three-dimensional structural analyses, but for most structures with well-separated periods and low damping, the result of the RSS and CQC methods will be nearly identical (this is true for most two-dimensional structural analyses).

The amplitude of displacement in each mode is dependent upon the spectral acceleration for that mode and its *modal participation factor*, which is a measure of the degree to which a certain mode participates in the response. The value of the modal participation factor depends on how the mode shapes are normalized, and so will not be given here, however the values are smaller for the higher modes with the result that the displacements for the higher modes are generally smaller than those of the first mode. The modal displacements are presented in Table 1-4 (to three decimal places, which is why some values are shown as zero) and plotted in Figure 1-9 for the first two modes as well as the RSS value. In this example, the influence of the two highest modes is very small and has been omitted from the diagram. It is difficult to distinguish between the first mode displacements and the RSS displacements in Figure 1-9; this is characteristic of structures with periods less than about 1 second, such as would be the case for most masonry structures.

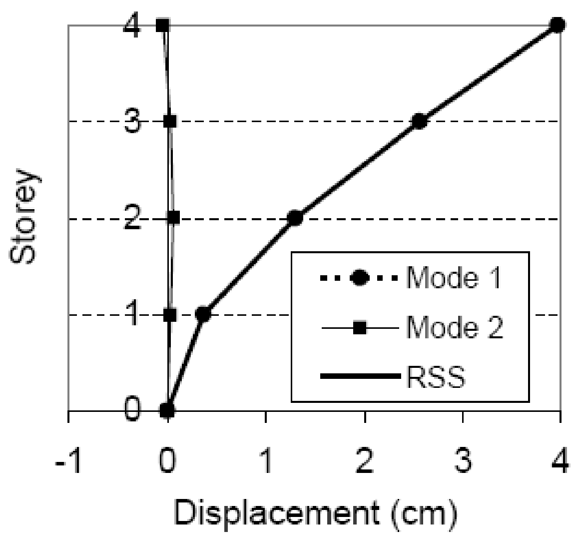


Figure 1-9. Modal displacements.

Modal analysis gives the modal shears and bending moments in each member and these values can be used to generate the shear and moment diagrams. These are summarized in Tables 1-5 and 1-6, and are graphically presented in Figure 1-10. Only the results from the first two modes are shown as the higher modes contribute very little to the response. Except for some contribution to the shears, the second mode is insignificant in contributing to the total values calculated using the RSS method.

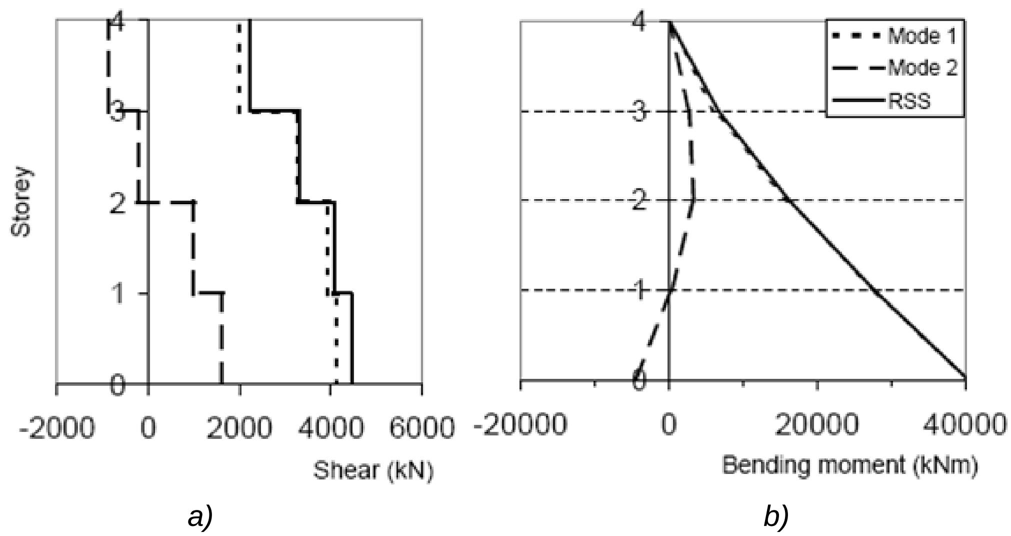


Figure 1-10. Modal analysis results: a) shear forces; b) bending moments.

The inertia force at each floor for each mode can be determined by taking the difference between the shear force above and below the floor in question. Modal inertia forces along with the RSS values are summarized in Table 1-7, and show that the higher modes at some levels contribute more than the first mode. Note that the sum of the inertia forces for each mode is equal to the base shear for that mode. However, the sum of the RSS values of the floor forces at each level is 6284 kN (obtained by adding values for storeys 1 to 4 in the last column of the table); this is not equal to the total base shear of 4468 kN found by taking the RSS of the base shears in each mode (see Table 1-3). This demonstrates the key rule in combining modal responses: **only primary quantities from each mode should be combined**. For example, if the designer is interested in the shear force diagram for the structure, it is necessary to find the shear forces in each mode and then combine these modal quantities using the RSS method. It is incorrect to find the total floor forces at each level from the RSS of individual modal values, and then use these total forces to draw the shear diagram. Even interstorey drift ratios, defined as the difference in the displacement from one floor to the next divided by the storey height, should be calculated for each mode and then combined using the RSS procedure. It would be incorrect to divide the total floor displacements by the storey height; although in this example since the deflection is almost entirely given by the first mode this approach would be very close to that found using the RSS method.

One of the disadvantages of modal analysis is that the signs of the forces are lost in the RSS procedure and so equilibrium of the final force system is not satisfied. Equilibrium is satisfied in each mode, but this is lost in the procedure to combine modal quantities since each quantity is squared. That is why it is important to determine quantities of interest by combining only the original modal values.

1.4.4.4 Comparison of static and modal analysis results

The equivalent static force analysis procedure, which will be presented in more detail in Section 1.5.4, has been applied to the four storey structure described above for the spectrum shown in Figure 1-8. Table 1-8 compares the results of the two types of analyses. It can be seen that both the base shear and moment given by the modal analysis method is about 75% of that given by the static method. This occurs with short period MDOF structures that respond in essentially the first mode because the modal mass of the first mode for walls is about 70 to 80% of the total mass. The top displacement from the modal analysis is 78% of the static displacement, nearly the same as the ratio of the base moments; this would be expected given that the deflection is mostly tied to the moment.

If the structure is a single-storey, SDOF system, the two analyses methods will give the same result. But for MDOF systems, such as two-storey or higher buildings, dynamic analysis will generally result in smaller forces and displacements than the static procedure.

The floor forces from the two analyses are quite different. The floor forces in the upper storeys obtained by modal analysis are less than the static forces, but in the lower storeys, a reverse trend can be observed. The reason for this is the contribution of the higher modes to the floor forces. It can be seen in Table 1-7, that at the 2^{nd} storey, the second mode contribution is the largest of all the modes. To ensure the required safety level when seismic design is performed using the equivalent static analysis procedure, NBCC 2005 seismic provisions (e.g. Clause 4.1.8.15) provides additional guidance on the level of floor forces to be used in connecting the floors to the lateral load resisting elements.

Table 1-1. Modal Periods and Masses

Mode	Period (sec)	Modal mass/ Total mass
1	0.400	0.696
2	0.062	0.210
3	0.022	0.070
4	0.012	0.024
Sum		1.000

Table 1-2. Mode Shapes

Storey	Mode Shapes			
	1st mode	2nd mode	3rd mode	4th mode
0	0.000	0.000	0.000	0.000
1	0.093	0.505	1.000	-1.000
2	0.328	1.000	0.334	0.969
3	0.647	0.544	-0.972	-0.619
4	1.000	-0.727	0.427	0.175

Note: mode shapes are normalized to a maximum of 1

Table 1-3. Spectral Accelerations, S_a, and Base Shears

Mode	Period (sec)	Spectral Acceleration S_a (g)	Modal mass / Total mass	Base Shear (kN)
1	0.400	0.74	0.696	4127
2	0.062	0.96	0.210	1617
3	0.022	0.96	0.070	534
4	0.012	0.96	0.024	184
Total base shear			ABS	6462
Total base shear			RSS	4468

Note: total weight = 8000 kN

Table 1-4. Modal Displacements

Storey	Modal Displacements (cm)				RSS
	1st mode	2nd mode	3rd mode	4th mode	
Base	0.000	0.000	0.000	0.000	0.00
1	0.367	0.021	0.002	0.000	0.37
2	1.300	0.042	0.001	0.000	1.30
3	2.564	0.023	-0.002	0.000	2.56
4	3.963	-0.031	0.001	0.000	3.96

Table 1-5. Modal Shear Forces

Storey	Shear Forces (kN) 1st mode	2nd mode	3rd mode	4th mode	RSS
0-1	4127	1617	534	-184	4468
1-2	3942	999	-143	204	4074
2-3	3287	-224	-369	-172	3320
3-4	1996	-888	289	68	2205

Table 1-6. Modal Bending Moments

Storey	Bending Moments (kNm) 1st mode	2nd mode	3rd mode	4th mode	RSS
Base	40053	-4511	-931	255	40320
1	27675	339	670	-298	27686
2	15849	3335	240	313	16201
3	5988	2665	-867	-204	6614
4	0	0	0	0	0

Table 1-7. Modal Inertia Forces (Floor Forces)

Storey	Floor Forces (kN) 1st mode	2nd mode	3rd mode	4th mode	RSS
1	185	618	677	-388	1012
2	655	1223	226	376	1455
3	1291	665	-658	-240	1612
4	1996	-888	289	68	2205
Sum	4127	1617	534	-184	4468

Table 1-8. Comparison of Static and Dynamic Analyses Results

Storey	Shear Forces (kN)		Floor Forces (kN)		Moments (kNm)		Deflections (cm)	
	Static	Modal(1)	Static	Modal(2)	Static	Modal(3)	Static	Modal(4)
Base			0	0	53280	40320	0	0
	5920	4468						
1			592	1012	35520	27686	0.48	0.37
	5328	4074						
2			1184	1455	19536	16201	1.70	1.30
	4144	3320						
3			1776	1612	7104	6614	3.32	2.56
	2368	2205						
4			2368	2205	0	0	5.11	3.96

Notes: (1) see Table 1-5, last column
(2) see Table 1-7, last column;
(3) see Table 1-6, last column;
(4) see Table 1-4, last column.

1.5 Seismic Analysis According to NBCC 2005

This section presents and explains the relevant seismic code provisions in NBCC 2005. Reference will be made here to NBCC 1995 where appropriate, but Appendix A contains the pertinent 1995 code provisions and a comparison of the design forces from the two codes.

1.5.1 Seismic Hazard

4.1.8.4.(6)

One of the major changes to the seismic provisions between the 1995 and 2005 editions of the NBCC is related to the determination of the seismic hazard. The 1995 code was based on probabilistic estimates of the peak ground acceleration and peak ground velocity for a probability of exceedance of 1/475 per annum (10% in 50 years). For NBCC 2005, the seismic hazard is based on a 2% in 50 years probability (corresponding to 1/2475 per annum), and it is represented by the 5% damped spectral response acceleration, $S_a(T)$. During the NBCC 2005 code development cycle, records became available, and the ability to compute how response spectral values vary with magnitude and distance from source to site greatly improved. Thus, it was possible to compute probabilistic estimates of spectral acceleration for different structural periods, and construct a response spectrum where each point on the spectrum has the same probability of exceedance. Such a spectrum is termed a *Uniform Hazard Spectrum*, or UHS. The acceleration UHS for Montreal is shown in Figure 1-11.

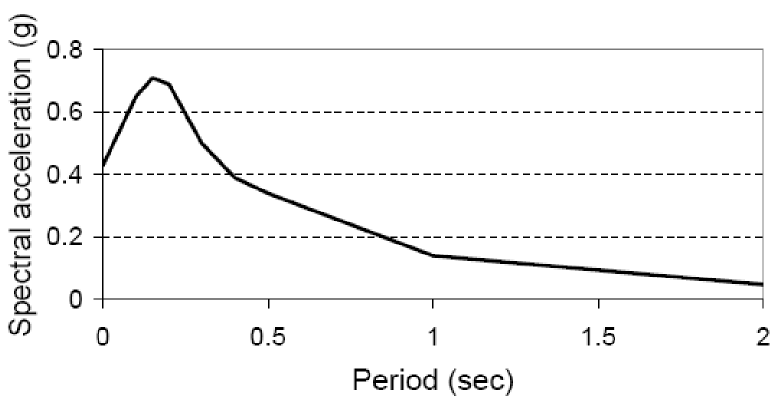


Figure 1-11. Uniform hazard spectrum for Montreal (UHS), 2% in 50 years probability, 5% damping.

For design purposes, the NBCC 2005 does not use the UHS, but rather an approximation given by four period-spectral values which are used to construct a spectrum, $S_a(T)$, which is used as the basis for the design spectrum. For many locations in the country, these values are specified in Table C-2, Appendix C to the NBCC 2005, along with the peak ground acceleration (PGA) for each location, which is used mainly for geotechnical purposes. For other Canadian locations, it is possible to find the values online at:

http://earthquakescanada.nrcan.gc.ca/hazard/interpolator/index_e.php

by entering the coordinates (latitude and longitude) of the location. The program does not directly calculate the $S_a(T)$ values, but instead, interpolates them from the known values at several surrounding locations. For detailed information on the models used as the basis for the NBCC 2005 seismic hazard provisions, the reader is referred to Adams and Halchuk (2003).

Figure 1-12 shows the $S_a(T)$ spectrum for Montreal and the corresponding UHS. Since $S_a(T)$ is constructed using only four points (corresponding to different periods), it is an approximation to the UHS, and it also reflects some conservatism in the code. At very short periods $S_a(T)$ is taken to be constant at the $S_a(0.2)$ value, and it does not decrease to the PGA, which is the UHS value at zero period. This may appear to be very conservative, but only a few structures have periods less than 0.2 sec, and there are other reasons related to the inelastic response of such short-period structures, to be conservative in this region. Note that many low-rise masonry buildings may have a fundamental period on the order of 0.2 sec.

The data needed to calculate the UHS values for large periods (over 2 seconds) is not available for all regions in Canada, and so between 2 seconds and 4 seconds, $S_a(T)$ is assumed to vary as $1/T$. Beyond 4 seconds there is even less data, and $S_a(T)$ is assumed to be constant at the $S_a(4)$ value for periods larger than 4 seconds. $S_a(T)$ is defined as the design hazard spectrum for sites located on what is termed soft rock or very dense soil. For sites situated on either harder rock or softer soil the hazard spectrum needs to be modified as discussed below.

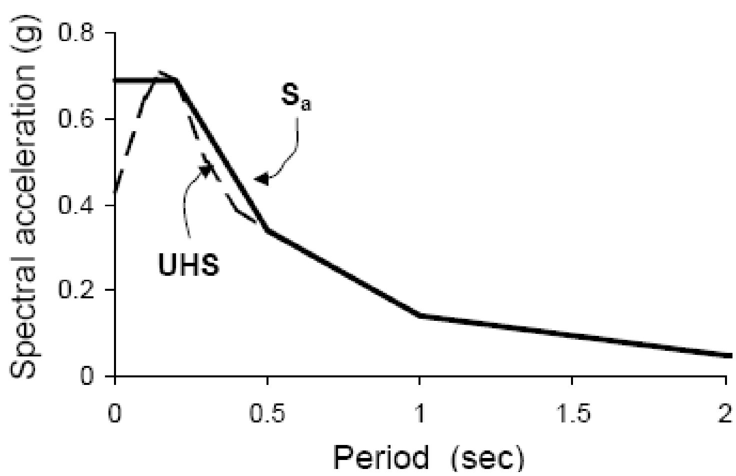


Figure 1-12. $S_a(T)$ and UHS spectrum for Montreal.

1.5.2 Effect of Site Soil Conditions

4.1.8.4

In the NBCC 2005, the seismic hazard given by the $S_a(T)$ spectrum has been developed for a site that consists of either very dense soil or soft rock (Site class C within NBCC 2005). If the structure is to be located on soil that is softer than this, the ground motion may be amplified, or in the case of rock or hard rock sites, the motion will be de-amplified. In NBCC 2005 two site coefficients are provided to be applied to the $S_a(T)$ spectrum to account for these local ground conditions. The coefficients depend on the building period, level of seismic hazard, as well as on the site properties, which are described in terms of site classes. The NBCC 2005 site coefficients are more detailed than the foundation factor, F, provided in previous code editions, but should better represent the effect of the local soil conditions on the seismic response.

Table 1-9 excerpted from NBCC 2005, describes six site classes, labelled from A to E, which correspond to different soil profiles (note that the seventh class, F, is one that fits none of the first six and would require a special investigation). The site classes are based on the properties of the soil or rock in the top 30 m. Site class C is the base class for which the site coefficients are unity, i.e. it is the type of soil on which the data used to generate the $S_a(T)$ spectrum is based. The table identifies three soil properties that can be used to identify the site class; the best one being the average shear wave velocity, $\overline{V}_s$, which is a parameter that directly affects the dynamic response. The site class determination is based on the weighted average, of the property being considered, in the top 30 m, which for $\overline{V}_s$ would correspond to the average velocity it would take for a shear wave to traverse the 30 m depth. NBCC 2005 and Commentary J (NRC, 2006) do not discuss the level from which the 30 m should be measured. For buildings on shallow foundations, the 30 m should be measured from the bottom of the foundation. However, if the building has a very deep foundation where the ground motion forces transferred to the building may come from both friction at the base and soil pressures on the sides, the answer is not so clear and may require a site specific investigation to determine the accelerations of the building foundation.

Table 1-9. NBCC 2005 Site Classification for Seismic Response (NBCC 2005 Table 4.1.8.4.A)

Site Class	Ground Profile Name	Average Properties in Top 30 m, as per Appendix A		
		Average Shear Wave Velocity, $\overline{V}_s$ (m/s)	**Average Standard Penetration Resistance, $\overline{N}_{60}$**	**Soil Undrained Shear Strength, s_u**
A	**Hard rock**	$\overline{V}_s > 1500$	Not applicable	Not applicable
B	**Rock**	$760 < \overline{V}_s \le 1500$	Not applicable	Not applicable
C	**Very dense soil and soft rock**	$360 < \overline{V}_s < 760$	$\overline{N}_{60} > 50$	$s_u > 100$kPa
D	**Stiff soil**	$180 < \overline{V}_s < 360$	$15 \le \overline{N}_{60} \le 50$	$50 < s_u \le 100$kPa
E	**Soft soil**	$\overline{V}_s < 180$	$\overline{N}_{60} < 15$	$s_u < 50$kPa
		Any profile with more than 3 m of *soil* with the following characteristics: ▪ plasticity index: PI > 20 ▪ moisture content: $w \ge 40\%$; and ▪ undrained shear strength: $s_u < 25$ kPa		
F	**Other soils**(1)	Site-specific evaluation required		

Notes:

(1) Other *soils* include:

a) liquefiable *soils*, quick and highly sensitive clays, collapsible weakly cemented *soils*, and other *soils* susceptible to failure or collapse under seismic loading,
b) peat and/or highly organic clays greater than 3 m in thickness,
c) highly plastic clays (PI>75) more than 8 m thick,
d) soft to medium stiff clays more than 30 m thick.

The effect of the site class on the response spectrum is given by the following two site coefficients: F_a, which modifies the spectrum $S_a(T)$ in the short period range (see Table 1-10), and F_v, which modifies $S_a(T)$ in the longer period range (see Table 1-11).

Table 1-10. Values of F_a as a Function of Site Class and $S_a(0.2)$ (NBCC 2005 Table 4.1.8.4.B)

Site Class	Values of F_a				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) = 1.25$
A	0.7	0.7	0.8	0.8	0.8
B	0.8	0.8	0.9	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0
D	1.3	1.2	1.1	1.1	1.0
E	2.1	1.4	1.1	0.9	0.9
F	(1)	(1)	(1)	(1)	(1)

Notes: (1) See Sentence 4.1.8.4.(5).

Table 1-11. Values of F_v as a Function of Site Class and $S_a(1.0)$ (NBCC 2005 Table 4.1.8.4.C)

Site Class	Values of F_v				
	$S_a(1.0) \leq 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \geq 0.5$
A	0.5	0.5	0.5	0.6	0.6
B	0.6	0.7	0.7	0.8	0.8
C	1.0	1.0	1.0	1.0	1.0
D	1.4	1.3	1.2	1.1	1.1
E	2.1	2.0	1.9	1.7	1.7
F	(1)	(1)	(1)	(1)	(1)

Notes: [1] See Sentence 4.1.8.4.(5).

Note that the F_a and F_v values depend on the level of seismic hazard as well as on the site soil class. For soft soil sites (site classes D and E), motion from a high hazard event would lead to higher shear strains in the soil, which gives rise to higher soil damping and reduced surface motion, when compared to a low hazard motion. The softer the soil, as given by a higher site classification, the higher the site coefficients, except for a few F_a values at high hazard level. For rock and hard rock, the site coefficients will generally be less than unity.

The F_a and F_v factors are applied to the $S_a(T)$ spectrum to give $S(T)$, which is the design spectral acceleration for the site. The calculation of $S(T)$ values will be illustrated with an example.

Figure 1-13 shows the design seismic hazard spectrum, $S(T)$, for Vancouver for a firm ground site, Class C, and a soft soil site, Class E. For Vancouver (Granville and 41 Ave): $S_a(0.2)$=0.96g, $S_a(0.5)$=0.66g, $S_a(1.0)$=0.34g, and $S_a(2.0)$=0.17 (see Appendix C, NBCC 2005; note that these values were taken from an earlier version of Table C-2 and are slightly different from the published values).

Interpolating from the values in Table 1-10 for site Class E and $S_a(0.2)$=0.96g, gives F_a=0.932, and from Table 1-11 for $S_a(1.0)$=0.34g, gives F_v=1.82.

The calculations to determine $S(T)$ for the Class E site are (see Clause 4.1.8.4.(6)):

For T=0.2 sec: $S(0.2) = F_a S_a(0.2) = 0.932 \times 0.96 = 0.89$ **S(0.2)=0.89**

For T=0.5 sec:

$S(0.5) = F_v S_a(0.5) = 1.82 \times 0.66 = 1.2$, or

$S(0.5) = F_a S_a(0.2) = 0.932 \times 0.96 = 0.89$, whichever is smaller

Since the smaller value governs, **S(0.5)=0.89**

For T=1 sec: $S(1.0) = F_v S_a(1.0) = 1.82 \times 0.34 = 0.62$ **S(1.0)=0.62**

For T=2 sec: $S(2.0) = F_v S_a(2.0) = 1.82 \times 0.17 = 0.31$ **S(2.0)=0.31**

For T≥4 sec: $S(T) = F_v S_a(2.0)/2 = 1.82 \times 0.17/2 = 0.155$ **S(T≥4.0)=0.155**

The resulting $S(T)$ soil Class C and E design spectra for Vancouver are plotted in Figure 1-13.

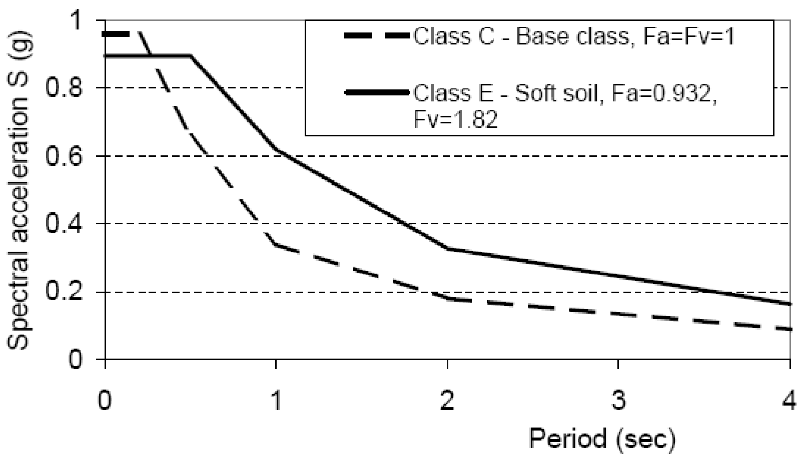


Figure 1-13. S(T) design spectra for Vancouver for site Classes C and E.

1.5.3 Methods of Analysis

4.1.8.7

NBCC 2005 prescribes two methods of calculating the design base shear of a structure. The *dynamic method* is the default method, but the *equivalent static method* can be used if the structure meets <u>any</u> of the following criteria:

(a) is located in a region of low seismic activity where $I_E F_a S_a(0.2) < 0.35$ (I_E is the earthquake importance factor of the structure as defined in Clause 4.1.8.5.(1)),

(b) is a regular structure less than 60 m in height with period, T_a, less than 2 seconds in either direction (T_a is defined as the fundamental lateral period of vibration of the structure in the direction under consideration, as defined in Clause 4.1.8.11.(3)),or

(c) is an irregular structure, but does not have Type 7 irregularity, and is less than 20 m in height with period, T_a, less than 0.5 seconds in either direction (see Section 1.5.10.1 for more details on irregularities).

The equivalent static method will be described in this section because it likely can be used on the majority of masonry buildings given the above criteria, and notwithstanding, if the dynamic method is used, it must be calibrated back to the base shear determined from the equivalent static analysis procedure. Basic concepts of the modal dynamic analysis method were presented in Section 1.4.4, and a further discussion is offered in Section 1.5.12.

1.5.4 Base Shear Calculations- Equivalent Static Analysis Procedure

4.1.8.11

The lateral earthquake forces used in design are specified in the NBCC 2005, and are based on the maximum (design) base shear, V, of the structure as given by Clause 4.1.8.11. The elastic base shear, V_e, denotes the base shear if the structure were to remain elastic. Design base shear, V, is equal to V_e reduced by the force reduction factors, R_d and R_o, (related to ductility and overstrength, respectively; discussed in Section 1.5.5), and increased by the importance factor I_E (see Table 1-12 for a description of parameters used in these relations), thus;

$$V = \frac{V_e I_E}{R_d R_o}$$

where

$$V_e = S(T_a) M_v W$$

represents the elastic base shear, M_v is a multiplier that accounts for higher mode shears, and W is the dead load, as defined in Table 1-12.

The relationship between V_e and V is shown in Figure 1-14. Note that the actual strength of the structure is greater than the design strength V.

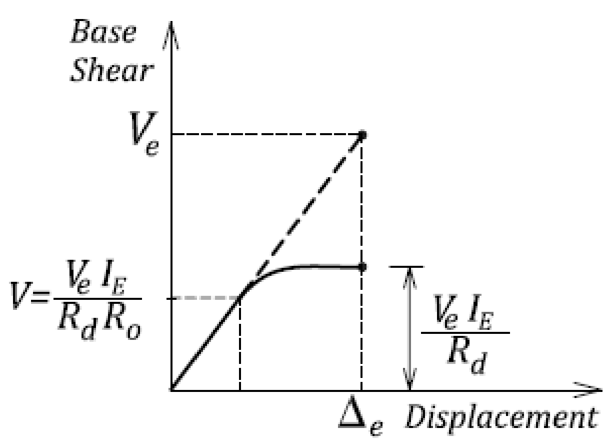


Figure 1-14. Design base shear, V, and elastic base shear, V_e.

NBCC 2005 prescribes the following lower and upper bounds for the design base shear, V:

<u>a) Lower bound:</u>
Because of uncertainties in the hazard spectrum, $S_a(T)$, for periods greater than 2 seconds, the minimum design base shear should not be taken less than:

$$V_{min} = \frac{S(2.0) M_v I_E W}{R_d R_o}$$

b) Upper bound:
Short period structures have small displacements, and there is not a huge body of evidence of failures for very low period structures, provided the structure has some ductile capacity. Thus an upper bound on the design base shear is given by:

$$V_{max} = \left(\frac{2S(0.2)}{3}\right)\left(\frac{I_E W}{R_d R_o}\right), \text{ provided } R_d \geq 1.5$$

M_v is not included in the above equation as $M_v = 1$ for short periods.

Some site specific studies for soil classes E and F, especially those located in high seismic zones, may show spectral values for periods of 0.5 to1.0 seconds to be greater than $2S(0.2)/3$. If this occurs it is recommended that the spectral value used in the short period range not be less than maximum value at the longer period.

Note that the design base shear force, V , corresponds to the design force at the ultimate limit state, where the structure is assumed to be at the point of collapse. Consequently, seismic loads are designed with a load factor value of 1.0 when used in combination with other loads (e.g. dead and live loads; see Table 4.1.3.2, NBCC 2005). It is also useful to recall that while V represents the design base shear, individual members are designed using factored resistances, ϕR , and since the nominal resistance, R , is greater than the factored resistance, the actual base shear capacity will be approximately equal to VR_o , as shown in Figure 1-14.

T_a denotes the *fundamental period* of vibration of the building or structure in seconds in the direction under consideration (i.e. direction of seismic force). The fundamental period of wall structures is given in the NBCC 2005 by:

a) $T_a = 0.05(h_n)^{3/4}$, where h_n is the height of the building in metres (Cl.4.1.8.11.3 (c)), or

b) other established methods of mechanics, except that T_a should not be greater than 2.0 times that determined in (a) above (Sub Cl.4.1.8.11.3.(d)iii).

The code formula to calculate T_a in (a) is simpler than the corresponding NBCC 1995 equation, in that it is based solely on building height and not on the length of the walls, and the allowance for using a calculated T_a in (b) is usually more liberal than in NBCC 1995. The period given by the NBCC 2005 in (a) is a conservative (short) estimate based on measured values for existing buildings. Using method (b) will generally result in a longer period, with resulting lower forces, and should be based on stiffness values reflecting possible cracked sections and shear deformations. For the purpose of calculating deflections, there is no limit on the calculated period as a longer period results in larger displacements (a conservative estimate), but it should never be less than that period used to calculate the forces.

Table 1-12. NBCC 2005 Seismic Design Parameters

Design parameter		NBCC reference
$S(T)$ =	the design spectral acceleration that includes the site soil coefficients F_a and F_v (see Section 1.5.2) $S(T) = F_aS_a(0.2)$ for $T < 0.2$ s $= F_vS_a(0.5)$ or $F_aS_a(0.2)$ whichever is smaller for T= 0.5 s $= F_vS_a(1.0)$ for T = 1.0 s $= F_vS_a(2.0)$ for T = 2.0 s $= F_vS_a(2.0)/2$ for $T \geq 4.0$ s	Cl.4.1.8.4(6)
M_v =	higher mode factor (see Section 1.5.6)	Cl.4.1.8.11.(5) Table 4.1.8.11
I_E =	importance factor for the design of the structure: 1.5 for post-disaster buildings, 1.3 for high importance structures, including schools and places of assembly that could be used as refuge in the event of an earthquake, 1.0 for normal buildings, and 0.8 for low importance structures such as farm buildings where people do not spend much time. See Table 4.1.2.1 in NBCC 2005 Part 4 for more complete definitions of the importance categories. There are also requirements for the serviceability limit states for the different categories.	Cl.4.1.8.5(1) Table 4.1.8.5
W =	dead load plus some portion of live load that would move laterally with the structure (also known as seismic weight). Live loads considered are 25% of the design snow load, 60% of storage loads for areas used for storage, and the full contents of any tanks. This requirement is the same as in the NBCC 1995 except that minimum partition load that need not exceed 0.5 kPa, and that parking garages need not be considered as storage areas.	Cl.4.1.8.2
R_d =	ductility related force modification factor that represents the capability of a structure to dissipate energy through inelastic behaviour (see Table 1-13 and Section 1.5.5); *ranges from 1.0 for unreinforced masonry to 2.0 for moderately ductile masonry shear walls.*	Table 4.1.8.9
R_o =	overstrength related force modification factor that accounts for the dependable portion of reserve strength in the structure (see Table 1-13 and Section 1.5.5); *equal to 1.5 for all reinforced masonry walls.*	Table 4.1.8.9

1.5.5 Force Reduction Factors R_d and R_o

4.1.8.9

Table 1-13 (NBCC 2005 Table 4.1.8.9) gives the R_d and R_o values for the different types of masonry lateral load-resisting systems, which are termed the Seismic Force Resisting Systems, SFRS(s), by NBCC 2005 Cl.4.1.8.2. The SFRS is that part of the structural system that has been considered in the design to provide the lateral resistance to the earthquake forces and effects. In addition to providing the R_d and R_o values, Table 1-13 lists height limits for the different systems depending on the level of seismic hazard and importance factor, I_E.

Table 1-13. Masonry R_d and R_o Factors and General Restrictions[(1)] - Forming Part of Sentence 4.1.8.9(1) (Source: NBCC 2005 Table 4.1.8.9)

Type of SFRS	R_d	R_o	Height Restrictions (m) [(2)]				
			Cases where $I_EF_aS_a(0.2)$				Cases where $I_EF_vS_a(1.0)$ >0.3
			<0.2	≥0.2 to <0.35	≥0.35 to ≤0.75	>0.75	
Masonry Structures Designed and Detailed According to CSA S304.1							
Moderately ductile shear walls	2.0	1.5	NL	NL	60	40	40
Limited ductility shear walls	1.5	1.5	NL	NL	40	30	30
Conventional construction - shear walls	1.5	1.5	NL	60	30	15	15
Conventional construction - moment resisting frames	1.5	1.5	NL	30	NP	NP	NP
Unreinforced masonry	1.0	1.0	30	15	NP	NP	NP
Other masonry SFRS(s) not listed above	1.0	1.0	15	NP	NP	NP	NP

Notes: (1) See Article 4.1.8.10.
(2) NP = not permitted.
NL = system is permitted and not limited in height as an SFRS; height may be limited in other parts of the NBCC.
Numbers in this Table are maximum height limits in m.
The most stringent requirement governs.

Commentary

Table 1-13 identifies the following five SFRS(s) related to masonry construction:
1. Moderately ductile shear walls
2. Limited ductility shear walls
3. Conventional construction: shear walls and moment resisting frames
4. Unreinforced masonry
5. Other undefined masonry SFRS(s)

Note that moderately ductile shear walls are assigned the highest R_d value of 2.0, leading to the lowest design forces for masonry structures. The detailing requirements, given in CSA S304.1-04, are the most restrictive of all the masonry shear wall types, but the height limitations imposed by the NBCC 2005 are the most liberal, allowing structures up to 60 m in height (approximately 20 storeys) in moderately high seismic regions. This type of construction would normally only be used in taller structures, but is required for masonry SFRS(s) used in post-disaster buildings. Moderately ductile squat shear walls, those with a height-to-length ratio less than 1, are a separate class of moderately ductile shear walls. They are allowed higher shear resistance, and less restrictive requirements on the height-to-thickness ratio, when compared to regular moderately ductile walls.

Limited ductility shear walls and conventional construction shear walls both have $R_d = 1.5$. The limited ductility walls have more stringent detailing requirements than the conventional construction walls, but the height restrictions imposed by the NBCC 2005 are not as onerous. It is likely that the most common type of masonry shear wall construction used would be conventional construction walls.

Conventional construction moment-resisting frames are also allowed an $R_d = 1.5$, but are not permitted in moderately high seismic regions. CSA S304.1 does not discuss moment frames and they will not be discussed further here as they are rarely, if ever, used in masonry design.

Unreinforced masonry construction is only allowed where $I_E F_a S_a(0.2) < 0.35$, and is limited to a height of 15 m, except that they can go to a height of 30 m if $I_E F_a S_a(0.2) < 0.2$. Unreinforced masonry does not have a good record in past earthquakes and is assigned $R_d = R_o = 1.0$ values, as there is usually no ductility and brittle failures are a possibility.

The R_o factor in NBCC 2005 is an overstrength factor to account for the real resistance capacity of the structure when compared to the factored design resistance. It is made up of 3 components: i) $1/\phi = 1.18 \approx 1.2$, ii) a factor that accounts for the expected yield strength of the reinforcement above the specified yield strength, and iii) a factor of about 1.1 that recognizes that, because of restrictions on possible locations for the reinforcement in masonry walls, the amount of reinforcement is in most cases larger than that required. This results in an $R_o = 1.5$ after some rounding of the factors (Mitchell et al., 2003).

1.5.6 Higher Mode Effects (M_v factor)

4.1.8.11.(5)

In the determination of elastic base shear, V_e, only the first mode spectral value $S(T)$ is used. To account for the additional base shear that comes from the higher modes, the M_v factor is introduced. M_v depends on the type of SFRS, the fundamental period T_a, and the ratio $S_a(0.2)/S_a(2.0)$. The M_v values assigned by NBCC 2005 are presented in Table 1-14. A discussion about the base overturning reduction factor, J, (also tabled) is provided in Section 1.5.8.

Table 1-14. Higher Mode Factor, M_v, and Base Overturning Reduction Factor, $J^{(1)(2)}$ Forming Part of Sentence 4.1.8.11.(5) (NBCC 2005 Table 4.1.8.11)

$S_a(0.2)/S_a(2.0)$	Type of Lateral Resisting Systems	M_v		J	
		$T_a \le 1.0$	$T_a \ge 2.0$	$T_a \le 0.5$	$T_a \ge 2.0$
< 8.0	**Moment resisting frames or coupled walls (3)**	1.0	1.0	1.0	1.0
	Braced frames	1.0	1.0	1.0	0.8
	Walls, wall-frame systems, other systems (4)	1.0	1.2	1.0	0.7
≥ 8.0	**Moment resisting frames or coupled walls (3)**	1.0	1.2	1.0	0.7
	Braced frames	1.0	1.5	1.0	0.5
	Walls, wall-frame systems, other systems (4)	1.0	2.5	1.0	0.4

Notes:

(1) For values of M_v between fundamental lateral periods, T_a, of 1.0 and 2.0 s, the product $S(T_a) \cdot M_v$ shall be obtained by linear interpolation.

(2) Values of J between fundamental lateral periods, T_a, of 0.5 and 2.0 s shall be obtained by linear interpolation.

(3) A "coupled wall" is a wall system with coupling beams, where at least 66% of the base overturning moment resisted by the wall system is carried by the axial tension and compression forces resulting from shear in the coupling beams.

(4) For hybrid systems, values corresponding to walls must be used or a dynamic analysis must be carried out as per Article 4.1.8.12.

Commentary

For structures with periods T_a greater than 1.0 s (typically, buildings of 10 storeys or higher), the contribution of higher modes to the base shear becomes increasingly important. In the eastern part of Canada, where $S_a(0.2)/S_a(2.0) \ge 8.0$, and where the $S_a(T)$ spectrum decreases sharply with periods beyond 0.2 seconds, the spectral acceleration for the second and third modes can be high compared to the first mode, and thus, these modes make a substantial contribution to the base shear. In western Canada, where $S_a(0.2)/S_a(2.0) < 8.0$, the spectrum does not decrease as sharply with increasing period, and the higher mode shears are less important when compared to the first mode base shear. It can be noted from Table 1-14 that the M_v factor is largest for wall structures, ranging in value from 1.0 to 2.5. This is relevant for high-rise masonry wall structures, and arises because the modal mass for the higher modes is larger in wall structures than in frames, and because the difference in periods between the modes is larger in wall than in frame structures.

For periods ranging from 1 to 2 seconds, M_v increases but $S(T)$ decreases, and it is important to note that interpolation between the two periods should be done on the product $S(T) \cdot M_v$, and not on the individual terms.

Beyond periods of 2 seconds, M_v is assumed constant, although it theoretically could be larger. However, since V_e is conservatively based on the $S(2.0)$ spectral value, it is appropriate to use the 2 second value of M_v.

Higher mode effects also affect the overturning moments and the value of J; this will be discussed in Section 1.5.8.

1.5.7 Vertical Distribution of Seismic Forces

4.1.8.11.(6)

The total lateral seismic force, V, is to be distributed such that a portion, F_t, is assumed to be concentrated at the top of the building; the remainder $(V - F_t)$ is to be distributed along the height of the building, including the top level, in accordance with the following formula (see Figure 1-15):

$$F_x = (V - F_t) \cdot \frac{W_x h_x}{\sum_{i=1}^{n} W_i h_i}$$

where

F_x – seismic force acting at level x

F_t – a portion of the base shear to be applied, in addition to force F_n, at the top of the building

h_x – height from the base of the structure up to the level x (base of the structure denotes level at which horizontal earthquake motions are considered to be imparted to the structure - usually the top of the foundations)

W_x - a portion of seismic weight, W, that is assigned to level x; that is, the weight at level x which includes the floor weight plus a portion of the wall weight above and below that level.

According to NBCC 2005, Sentence 4.1.8.11.(4), the seismic weight W is the sum of the weights at each floor, $W = \sum_{1}^{n} W_i$ (see Table 1-12).

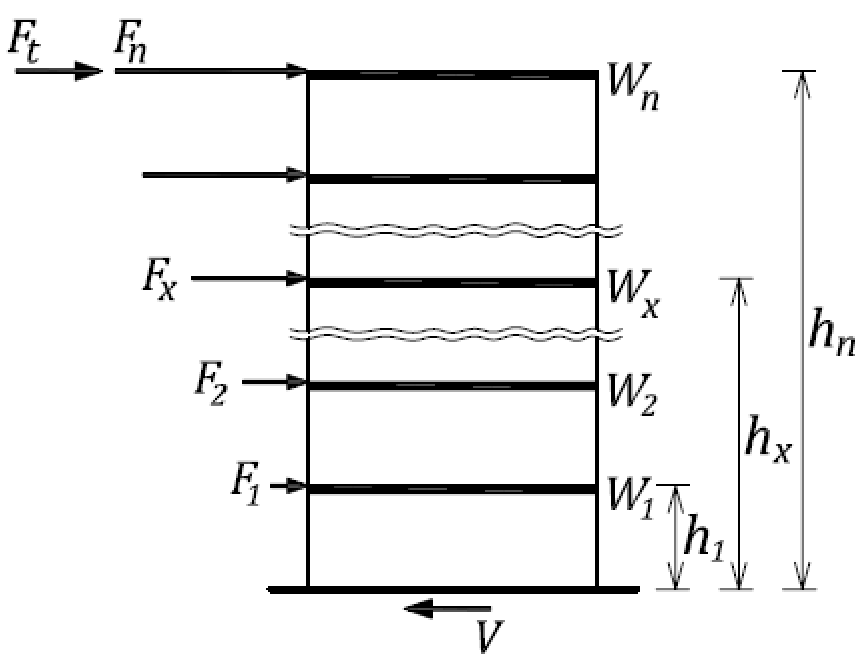


Figure 1-15. Vertical force distribution.

Commentary

The above formula for the force distribution is based on a linear first mode approximation for the acceleration at each level. The purpose of applying force F_t at the top of the structure is to increase the storey shear forces in the upper part of longer period structures where the first mode approximation is not correct. For periods less than 0.7 sec, shear is dominated by the first mode and so $F_t = 0$. The F_t force is determined as follows, see Cl.4.1.8.11.(6):

$F_t = 0$ for $T_a \le 0.7$ sec

$F_t = 0.07T_aV$ for $0.7 < T_a \le 3.6$ sec

$F_t = 0.25V$ for $T_a > 3.6$ sec

The remaining force, $V - F_t$, is distributed assuming the floor accelerations vary linearly with height from the base. By establishing the forces at each floor level, the total storey shears can be calculated using statics.

1.5.8 Overturning Moments (J factor)

4.1.8.11.(5)
4.1.8.11.(7)

While higher mode forces can make a significant contribution to the base shear, they make a much smaller contribution to the storey moments. Thus, moments at each storey level determined from the seismic floor forces, which include the higher mode shears in the form of the F_t factor, result in overturning moments that are too large. Previous editions of the NBCC have traditionally used a factor, termed the J factor, to reduce the moments, but the value of the J factor and how it is applied over the height of the structure is substantially different in NBCC 2005.

The J factor values are given in Table 1-14. Note that for the 2 second period, J is nearly equal to the inverse of M_v, which implies that the overturning moment at the base of the structure is governed by the first mode.

The overturning moment at any level shall be multiplied by the factor J_x (see Figure 1-16), where

$J_x = 1.0$ for $h_x \ge 0.6h_n$ (there is no reduction over the top 40% of the structure), and

$J_x = J + (1 - J)(h_x/0.6h_n)$ for $h_x < 0.6h_n$ (a linear increase from J at the base to 1.0 at the 60% level).

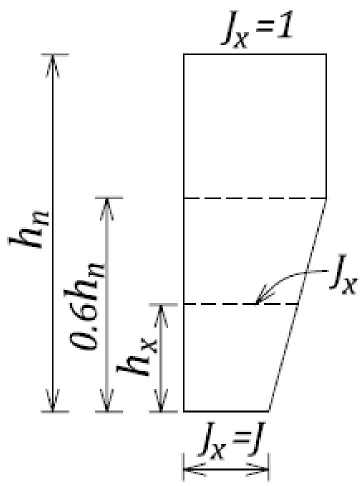


Figure 1-16. Distribution of the J_x factor over the building height.

Commentary

How the J factor and reduced overturning moments are incorporated into a structural analysis is not always straightforward, and it depends on the structural system.

For shear wall structures the overturning moments can be calculated using the floor forces from the lateral force distribution, and then reduced by the J_x factor at each level to give the design overturning moments. Without applying the J factor, the wall moment capacity would be larger, leading to higher shears when the structure yields, and could result in a shear failure.

For frames, the member shears, moments and axial loads, resulting from applying the lateral seismic forces at each floor level, will be too large. This would essentially result in higher axial loads in the columns, but not increase the shear demand on the structure, and so would be conservative. The J factor for frames is usually small, and it is believed that many designers ignore it as it is conservative to do so.

1.5.9 Torsion

1.5.9.1 Torsional effects

4.1.8.11.(8)

Torsional effects, that are concurrent with the effects of the lateral forces F_x, and that are caused by the following torsional moments shall be considered in the design of the structure:

a) torsional moments introduced by eccentricity between the centre of mass and the centre of resistance, and their dynamic amplification, or
b) torsional moments due to accidental eccentricities.

In determining the torsional forces on members the stiffness of the diaphragms is important. The discussion in Sections 1.5.9.1 to 1.5.9.3 considers rigid diaphragms only, while flexible diaphragms are discussed in Section 1.5.9.4.

Commentary

Torsional effects have been associated with many building failures during earthquakes. Torsional moments, or torques, arise when the lateral inertial forces acting through the centre of mass at each floor level do not coincide with the resisting structural forces acting through the centres of resistance. The *centre of mass*, C_M, is a point through which the lateral seismic inertia force can be assumed to act. The seismic shear is resisted by the vertical elements, and if the resultant of the shear forces does not lie along the same line of action as the inertia force acting through the centre of mass, then a torsional moment about a vertical axis will be created. The *centre of resistance*, C_R, also known as the centre of stiffness, is a point through which the resultant of all resisting forces act provided there is no torsional rotation of the structure. If the centre of mass at a certain floor level does not coincide with its centre of resistance, the building will twist in the horizontal plane about C_R. Torsion generates significant additional forces and displacements of the vertical elements (e.g. walls) furthest away from C_R. Ideally, C_R should coincide with, or be close to C_M, and sufficient torsional resistance should be available to keep the rotations small. Figure 1-17 shows two different plan configurations, one of which has a non-symmetric wall layout (a), and the other one with a symmetric layout (b). Both plans have approximately the same amount of walls in each direction but the symmetric building will perform better. The location of the shear walls determines the torsional stiffness of the structure; widely spaced walls provide high torsional stiffness and consequently small torsional rotations. Walls placed around the perimeter of the building, such as shown in Figure 1-17b, have very high torsional stiffness and are representative of low-rise or single-storey buildings. Taller buildings, which often have several shear walls distributed across the footprint of the structure, also give satisfactory torsional resistance (see Section 1.5.9.2 for a discussion on torsional sensitivity).

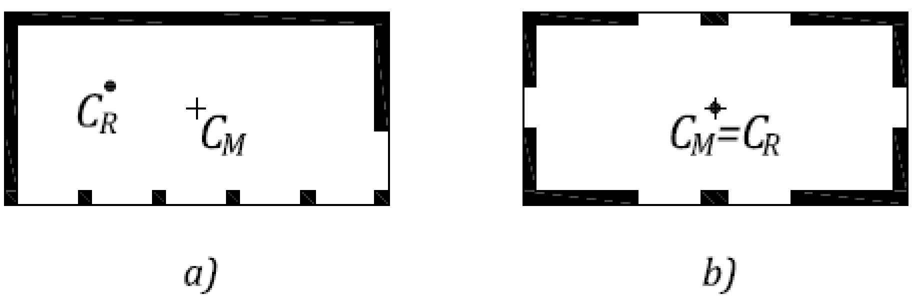


Figure 1-17. Building plan: a) non-symmetric wall layout (significant torsional effects);

b) symmetric wall layout (minor torsional effects).

Figure 1-18a shows a building plan (of a single storey building, or one floor of a multi-storey building), for which the centre of mass, C_M, and the centre of resistance, C_R, do not coincide. The distance between C_R (at each floor) and the line of action of the lateral force (at each floor), which passes through C_M is termed the *natural floor eccentricity*, e_x (note that the eccentricity is measured perpendicular to the direction of lateral load). The effect of the lateral seismic force, F_x, which acts at point C_M, can be treated as the superposition of the following two load cases: a force F_x acting at point C_R (no torsion, only translational displacements, see Figure 1-18b, and pure torsion in the form of torsional moment, T_x, about the point C_R, as shown in Figure 1-18c. The torsional moment, T_x, is calculated as the product of the floor force, F_x, and the eccentricity e_x.

In addition to the natural eccentricity, the NBCC requires consideration of an additional eccentricity, termed the *accidental eccentricity*, e_a. Accidental eccentricity is considered because of possible errors in determining the natural eccentricity, including errors in locating the centres of mass as well as the centres of resistance, additional eccentricities that might come from yielding of some elements, and perhaps from some torsional ground motion.

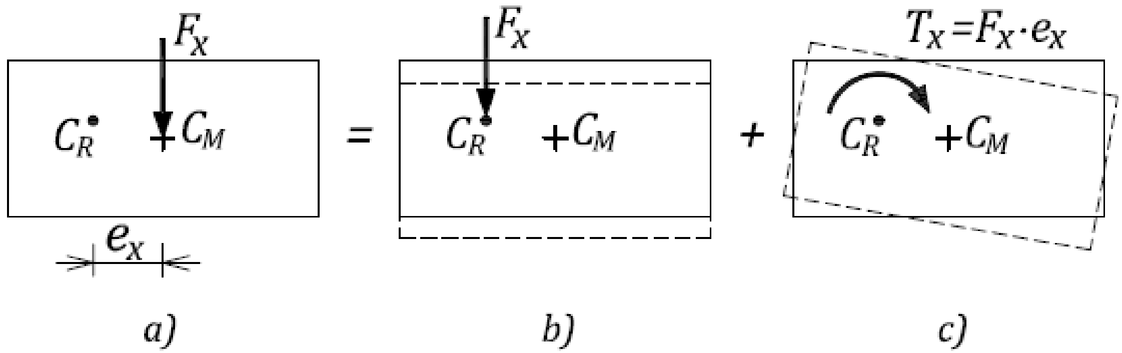


Figure 1-18. Torsional effects can be modelled as a combination of a seismic force, F_x, at point C_R (causing translational displacements only) and a torsional moment, T_x (causing rotation of building plan) about point C_R.

Finding the centre of resistance, C_R, may be a complex task in some cases. For single-storey structures it is possible to determine a centre of stiffness, which is the same as the C_R. However in multi-storey structures, C_R is not well defined. For a given set of lateral loads, it is possible to find the location on each floor through which the lateral load must pass, so as to produce zero rotation of the structure about a vertical axis. These points are often called the centres of rigidity, rather than centres of stiffness or resistance, but they are a function of the loading as well as the structure, and so centres of rigidity are not a unique structural property. A different set of lateral loads will give different centres of rigidity. Earlier versions of the NBCC required the determination of the C_R location so as to explicitly determine e_x, as it was necessary to amplify e_x (by factors of 1.5 or 0.5) to determine the design torque at each floor level. NBCC 2005 does not require this amplification, so the effect of the torque from the natural eccentricities can come directly from a 3-D lateral load analysis, without the additional work of explicitly determining e_x. However, NBCC 2005 requires a comparison of the torsional stiffness to the lateral stiffness of the structure to evaluate the torsional sensitivity, and so requires increased computational effort in this regard.

1.5.9.2 Torsional sensitivity

4.1.8.11.(9)

NBCC 2005 requires the determination of a torsional sensitivity parameter, B, which is used to determine possible analysis methods. To determine B, a set of lateral forces, F_x, is applied at a distance of $\pm 0.1 D_{nx}$ from the centre of mass C_M, where, D_{nx}, is the plan dimension of the building perpendicular to the direction of the seismic loading being considered. The set of lateral loads, F_x, to be applied can either be the static lateral loads or those determined from a dynamic analysis. A parameter, B_x, evaluated at each level, x, should be determined from the following equation (see Figure 1-19):

$$B_x = \frac{\delta_{\max}}{\delta_{ave}}$$

where

$\delta_{\max}$ - the maximum storey displacement at level x at one of the extreme corners, in the direction of earthquake, and

δ_{ave} - the average storey displacement, determined by averaging the maximum and minimum displacements of the storey at level x.

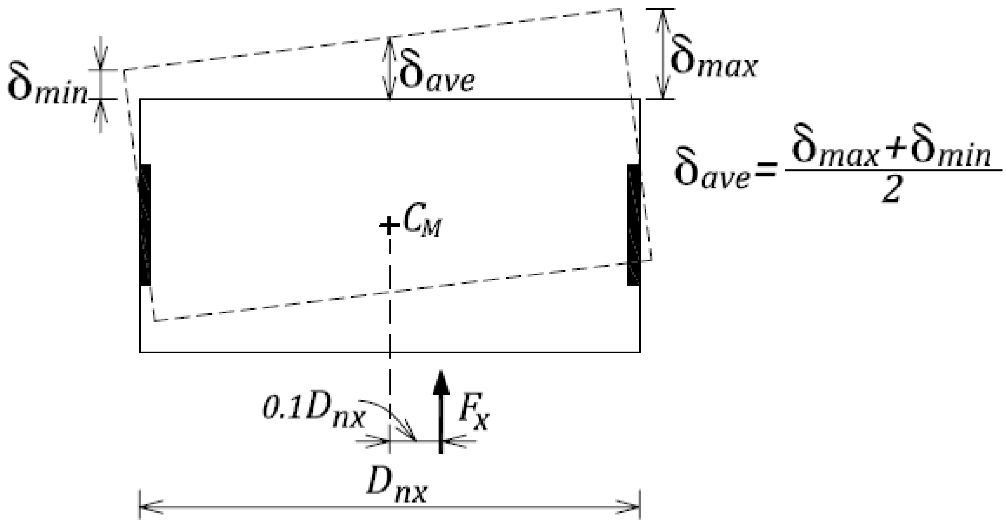


Figure 1-19. Torsional displacements used in the determination of B_x.

The torsional sensitivity, B, is the maximum value of B_x for all storeys for both orthogonal directions. Note that B_x needs not be considered for one-storey penthouses with a weight less than 10% of the level below.

Commentary

A structure is considered to be torsionally sensitive when the torsional flexibility compared to the lateral flexibility is above a certain level, that is, when $B > 1.7$. Torsionally sensitive buildings are considered to be torsionally vulnerable, and NBCC 2005 in some cases requires that the effect of natural eccentricity be evaluated using a dynamic analysis, while the effect of accidental eccentricity be evaluated statically.

Structures that are not torsionally sensitive, or located in a low seismic region where $I_E F_a S_a(0.2) < 0.35$, can have the effects of torsion evaluated using only the equivalent static analysis. If the structure is torsionally sensitive and located in a high seismic region, a dynamic analysis must be used to determine the effect of the natural eccentricity, but the accidental eccentricity effects must be evaluated statically, and the results then combined with the dynamic results, as discussed in Section 1.5.9.3. A static torsional analysis of the accidental eccentricity, on a torsionally flexible building, will lead to large torsional displacements, and generally to large torsional forces in the elements, and thus may require a change in the structural layout so that the structure is not so torsionally sensitive.

1.5.9.3 Determination of torsional forces

4.1.8.11.(10)

Torsional effects should be accounted for as follows:

a) if $B \leq 1.7$ (or $B > 1.7$ and $I_E F_a S_a(0.2) < 0.35$), the equivalent static analysis procedure can be used, and the torsional moments, T_x, about a vertical axis at each level throughout the building, should be considered separately for each of the following load cases:

i) $T_x = F_x(e_x + 0.1D_{nx})$, and

ii) $T_x = F_x(e_x - 0.1D_{nx})$.

The analysis required to determine the element forces, for both the lateral load and the above torques, is identical to that required to determine the B factor, where the lateral forces are applied at a distance $\pm 0.1D_{nx}$ from the centre of mass, C_M, as shown by the dashed arrows in Figure 1-20.

b) if $B > 1.7$, and $I_E F_a S_a(0.2) \geq 0.35$, the dynamic analysis procedure must be used to determine the effects of the natural eccentricities, e_x. The results from the dynamic analysis must be combined with those from a static torsional analysis that considers only the accidental torques given by

$T_x = +F_x(0.1D_{nx})$, or

$T_x = -F_x(0.1D_{nx})$

In this analysis, F_x can come from either the equivalent static analysis or from a dynamic analysis.

c) if $B \leq 1.7$, it is permitted to use a three-dimensional dynamic analysis with the centres of mass shifted by by a distance of $\pm 0.05D_{nx}$ (see Cl.4.1.8.12.(4)b).

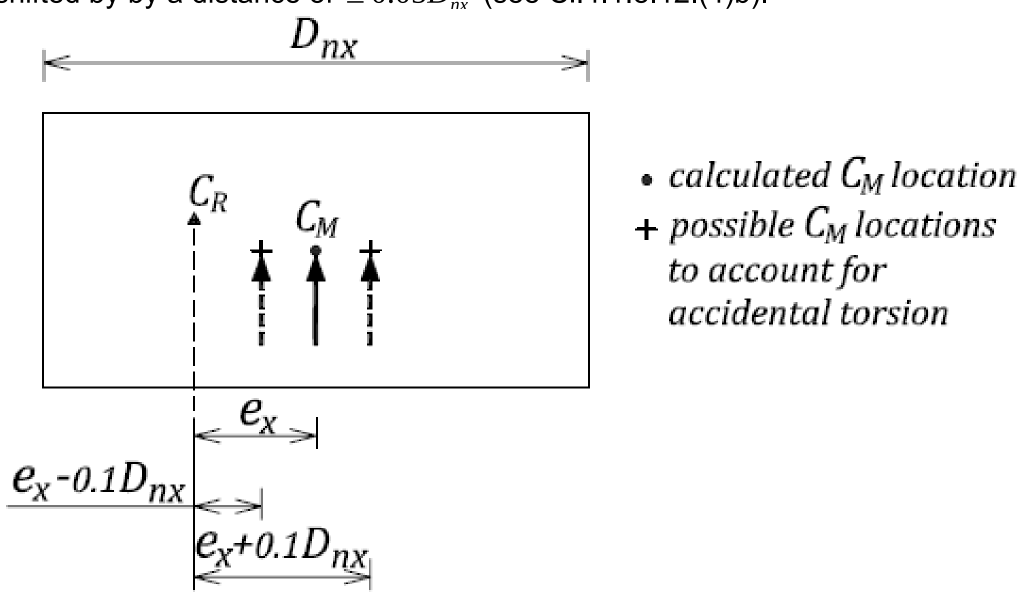


Figure 1-20. Torsional eccentricity according to NBCC 2005.

Commentary

When results from a dynamic analysis are combined with accidental torques that use the lateral forces F_x from the equivalent static procedure, the designer should ensure that the analysis is done in a consistent manner, that is, by using either the elastic forces or the reduced design forces (elastic forces modified by $I_E / R_d R_o$). The final force results should be given in terms of the reduced design forces, while the displacements should correspond to the elastic displacements.

If the structure is torsionally sensitive, $B > 1.7$, and if $I_E F_a S_a(0.2) \geq 0.35$, then the member forces and displacements from the accidental eccentricity must be evaluated statically by applying a set of torques to each floor of $\pm F_x(0.1D_{nx})$. The set of lateral forces, F_x, can come from either a static or a dynamic analysis. NBCC 2005 is mute on whether the set of lateral static forces should be scaled to match the dynamic base shear (if the dynamic base shear is larger than the static value), and whether the dynamic set should correspond to the set determined with the floor rotations restrained or not restrained (see Section 1.5.12). It is suggested here that if a set of static forces is used, they should (if necessary) be scaled up to match the base shear from the rotationally restrained dynamic analysis.

The static approach to determine member forces and displacements from the accidental eccentricity is illustrated in Figure 1-21.

If the static forces are to be used, then the following steps need to be followed:

1. The forces F_x are determined using the equivalent static method.
2. Torsional moments at each level are found using the following equations $T_x = +F_x(0.1D_{nx})$, or $T_x = -F_x(0.1D_{nx})$.
3. Displacements and forces due to torsional effects are determined, and combined with the results from the dynamic analysis. Note that, in buildings with larger periods, F_t will cause large rotations and displacements, and the results will probably be conservative.

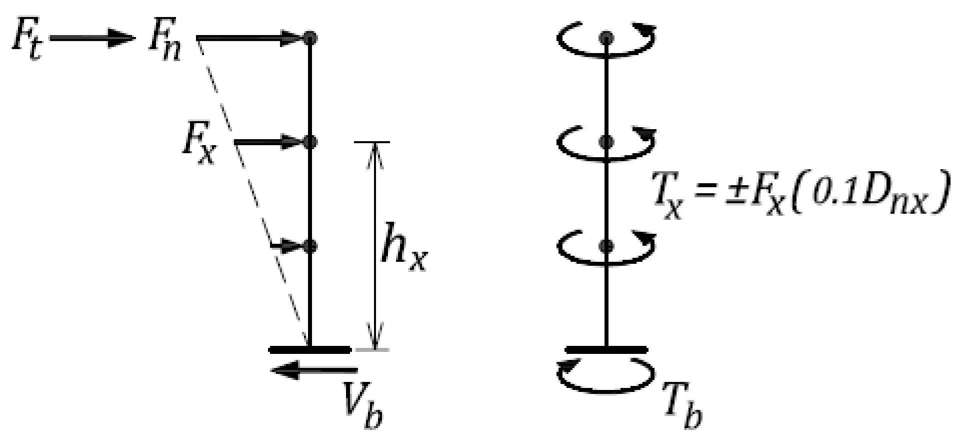


Figure 1-21. Static approach to determine the accidental eccentricity effects (Anderson, 2006).

If a dynamic set of floor forces, F_x, are to be used, they should be scaled, if necessary (as discussed in Section 1.5.12), to be equal to the design base shear. For the determination of the storey torques, the force F_x at each floor can be determined from the dynamic analysis by taking the difference in the total shear in the storeys above and below the floor in question. These floor forces are not necessarily the correct floor forces (as discussed in Section 1.4.4.3), however the sum of these forces equals the design base shear and they provide a reasonable set of lateral forces to use for the accidental eccentricity calculations. The second and third steps discussed in the previous paragraph are then the same.

If the structure is not torsionally sensitive ($B \leq 1.7$), and a dynamic analysis is being used, it is permissible to account for both the lateral forces and the torsional eccentricity, including the natural and accidental eccentricity, by using a 3-D dynamic analysis and moving the centre of mass by the distance $\pm 0.05D_{nx}$. This would require four separate analyses, two in each direction. In these dynamic analyses the accidental eccentricity is taken as $\pm 0.05D_{nx}$, while in the static application it is taken as $\pm 0.10D_{nx}$. It is thought that the real accidental eccentricity is about $\pm 0.05D_{nx}$, but it would likely be amplified during an earthquake; this is reflected in the results of a dynamic analysis. Thus, $\pm 0.10D_{nx}$ is used in the static case to account for accidental eccentricity and possible dynamic amplification.

When using a 3-D dynamic analysis for torsional response, it is important to correctly model the mass moment of inertia about a vertical axis. If the floor mass is entered as a point mass at the mass centroid, it will not have the correct mass moment of inertia and the torsional period will be too small. This will have the effect of making the structure appear to be torsionally stiffer than it really is, and could lead to smaller torsional deflections.

When applying the lateral loads in one direction, torsional response gives rise to forces in the elements in the orthogonal direction. For structures with lateral force resisting elements oriented along the principal orthogonal axes, NBCC 2005 Cl. 4.1.8.8.(1)a) requires independent analyses along each axis. For structures with elements oriented in non-orthogonal directions (as shown in Section 1.5.10.1 for Type 8 Irregularity), an independent analysis about any two orthogonal axes is sufficient in low seismic zones, but in higher zones, it is required that element forces from loading in both directions be combined. The suggested method for combining forces from both directions is the “100+30%” rule that requires the forces in any element that arise from 100% of the loads in one direction be combined with 30% of the loads in the orthogonal direction, see NBCC 4.1.8.8.(1)c). Another method is to apply the ‘root-sum-square’ procedure to the forces in each element from 100% of the loads applied in both directions. The two methods usually give close agreement and are based on the knowledge that the probability of the maximum forces from the two directions occurring at the same time is low. For some orthogonal systems, it is possible that the orthogonal forces from the effects of torsion are substantial, and a prudent design may consider combined forces from both directions as described above, especially in high seismic regions.

Note that the NBCC requirements are based on an estimate of the elastic properties of the structure. When the structure yields, the eccentricity between the inertia forces acting through the centres of mass and the resultant of the resisting forces based on the capacity of the members, termed the plastic eccentricity, will be different than the elastic eccentricity. In most cases, the plastic eccentricity will be less than the elastic eccentricity. However, there may be cases where some elements are stronger than necessary and do not yield; this could increase the eccentricity when other elements yield, and it should be avoided if possible.

1.5.9.4 Flexible diaphragms

Diaphragms are horizontal elements of the SFRS whose primary role is to transfer inertial forces throughout the building to the vertical elements (shear walls in case of masonry buildings) that resist these forces. A diaphragm can be treated in a manner analogous to a beam lying in a horizontal plane where the floor or roof deck functions as the web to resist the shear forces, and the boundary elements (bond beams in case of masonry buildings) serve as the flanges in resisting the bending moment. How the total shear force is distributed to the vertical elements of the SFRS will depend on their rigidity compared to the rigidity of the diaphragm. For design purposes, diaphragms are usually classified as rigid or flexible, but that very much depends on the type of structure. Structures with many walls and small individual diaphragms between the walls clearly can be considered as having flexible diaphragms. In large plan structures, such as warehouses or industrial buildings with the SFRS members located around the perimeter, it is more common to assume the diaphragm as being rigid. However the flexibility of the diaphragm may lead to a considerable increase in the period of the structure, and lead to deformations considerably larger than the deformations of the SFRS, in which case a more complex analysis would be required.

In *rigid diaphragms*, shear forces are distributed to vertical elements in proportion to their stiffness. Torsional effects are considered following the approach outlined in Sections 1.5.9.1 to 1.5.9.3. Concrete diaphragms, or steel diaphragms with concrete infill, are usually considered rigid.

In *flexible diaphragms*, distribution of shear forces to vertical elements is independent of their relative rigidity; these diaphragms act like a series of simple beams spanning between vertical elements. A flexible diaphragm must have adequate strength to transfer the shear forces to the SFRS members, but cannot distribute torsional forces to the SFRS members acting at right angles to the direction of earthquake motion without undergoing unacceptable displacements. Corrugated steel diaphragms without concrete fill, and wood diaphragms, are generally considered flexible; however, steel and wood diaphragms with horizontal bracing could be considered rigid.

Figure 1-22a shows the plan view of a simple one storey structure with walls on three sides and non-structural glazing on the fourth side. For an earthquake producing an inertia force, V , the walls provide resisting forces to the diaphragm as shown. The displacement of the diaphragm would be as shown in Figure 1-22b, and it is the size of the displacements that determines whether the diaphragm is considered flexible or rigid. If the displacements are too large to be acceptable, the diaphragm would be classed as flexible, and cannot be used with such a layout of the SFRS. In general, flexible diaphragms require that the SFRS has at least two walls in each direction such as shown in Figure 1-17b.

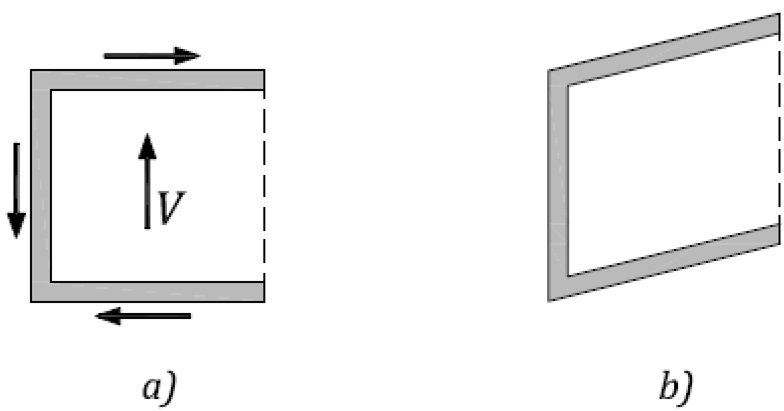


Figure 1-22. Building plan: a) loads on diaphragm; b) displaced shape of a flexible diaphragm.

In determining how the inertia forces are distributed to the SFRS, the flexible diaphragm should be divided into sections, with each section bounded by two walls in the direction of the inertia forces; preferably these two walls will be located on the sides of the section. The inertia forces from each section are then distributed to the SFRS on the basis of tributary areas. Equilibrium must be satisfied, and the diaphragm must have sufficient strength in shear and bending to act as a horizontal beam carrying the loads to the supports. Figure 1-23 shows a flexible roof system supported by three walls in the N-S direction. The roof should be divided into two sections as shown, with the inertia force from section 1 distributed to walls A and B. Section 2 must be considered as a beam with a cantilever end extending beyond wall C. Equilibrium of section 2 then gives rise to a high force in Wall C, with the overhanging portion contributing to a reduction in the force in wall B.

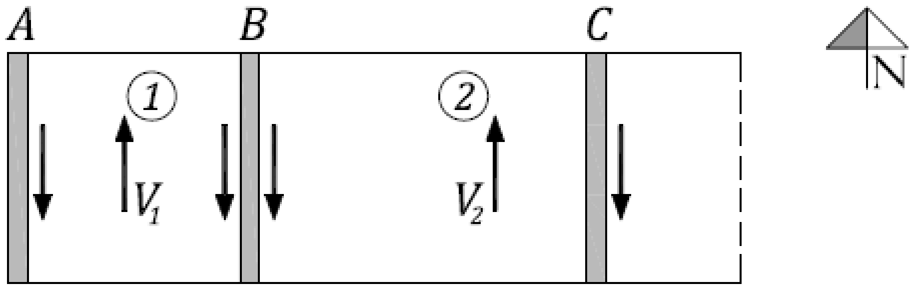


Figure 1-23. Plan view for analysis of flexible diaphragm.

NBCC 2005 requires that accidental eccentricity be considered. With rigid diaphragms it is clear how this can be accomplished, as described in the above sections, but trying to account for accidental eccentricity in flexible diaphragms raises several questions about how it is to be applied. NBCC 2005 Commentary J, paragraph 179 (NRC, 2006) states that it is sufficient to consider an eccentricity of $\pm 0.05D_{nx}$, where D_{nx} is defined as the width of the building in the direction perpendicular to the direction of the earthquake motion. If the structure consists of a single roof section with supporting walls at each end separated by the distance D_{nx}, moving the centre of mass by $0.05D_{nx}$ would increase the wall reactions by 10%, and the accidental eccentricity requirement would be satisfied. For a structure with several walls in the direction of the earthquake motion, shifting the centre of mass by $\pm 0.05D_{nx}$, which would require moving the centre of mass of each section by this amount, could lead to unrealistic situations, as well as requiring a considerable increase in computational effort. Even flexible diaphragms will have some stiffness, and in many cases will transfer some of the torsional load to the walls perpendicular to the direction of motion. This transfer is ignored when designing for flexible diaphragms, but does provide extra torsional resistance. It is suggested that the wall forces determined without any accidental eccentricity all be increased by 10% to account for the accidental eccentricity. This minimizes the number of calculations required, and it is suggested that it satisfies the intent of NBCC 2005.

1.5.10 Configuration Issues: Irregularities and Restrictions

1.5.10.1 Irregularities

4.1.8.6

New definitions of structural irregularities represent a substantial change in NBCC 2005. There are eight different types of irregularity, and these are used to trigger restrictions and special requirements, some of which are more restrictive than those in previous codes.

Table 1-15 (same as Table 4.1.8.6 in NBCC 2005) lists the eight types of irregularity, and the notes to the table refer to the relevant code clauses that consider the irregularity. The NBCC 2005, Commentary J (NRC, 2006) provides an expanded description of each type of irregularity. If a structure has none of the listed irregularities it is considered to be a *regular structure*. A trigger for the NBCC 2005 irregularity provisions (Cl.4.1.8.6) is the presence of one of eight types of irregularity in combination with the higher seismic hazard index, that is, $I_E F_a S_a(0.2) > 0.35$.

Table 1-15. Structural Irregularities[1] Forming Part of Sentence 4.1.8.6.(1) (NBCC Table 4.1.8.6.)

Type	Irregularity Type and Definition	Notes
1 ***Vertical stiffness irregularity***	Vertical stiffness irregularity shall be considered to exist when the lateral stiffness of the SFRS in a *storey* is less than 70% of the stiffness of any adjacent *storey*, or less than 80% of the average stiffness of the three *storeys* above or below.	(2) (3) (4)
2 ***Weight (mass) irregularity***	Weight irregularity shall be considered to exist where the weight, W_i, of any *storey* is more than 150% of the weight of an adjacent *storey*. A roof that is lighter than the floor below need not be considered.	(2)
3 ***Vertical geometric irregularity***	Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the SFRS in any *storey* is more than 130 percent of that in an adjacent *storey*.	(2) (3) (4) (5)
4 ***In-plane discontinuity in vertical lateral force-resisting element***	An in-plane offset of a lateral-force-resisting element of the SFRS or a reduction in lateral stiffness of the resisting element in the *storey* below.	(2) (3) (4) (5)
5 ***Out-of-plane offsets***	Out-of-plane offsets are discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements of the SFRS.	(2) (3) (4) (5)
6 ***Discontinuity in capacity - weak storey***	A weak storey is one in which the storey shear strength is less than that in the storey above. The *storey* shear strength is the total strength of all seismic-resisting elements of the SFRS sharing the *storey* shear for the direction under consideration.	(3)
7 ***Torsional sensitivity***	Torsional sensitivity shall be considered when diaphragms are not flexible, and when the ratio B>1.7 (see Sentence 4.1.8.11(9)).	(2) (3) (4) (6)
8 ***Non-orthogonal systems***	A non-orthogonal system irregularity shall be considered to exist when the SFRS is not oriented along a set of orthogonal axes.	(4) (7)

Notes: (1) One-*storey* penthouses with a weight less than 10% of the level below need not be considered in the application of this table.
(2) See Article 4.1.8.7.
(3) See Article 4.1.8.10.
(4) See Appendix A.
(5) See Article 4.1.8.15.
(6) See Sentences 4.1.8.11.(9), (10), and 4.1.8.12.(4)
(7) See Article 4.1.8.8.

Commentary

The equivalent static analysis procedure is based on a regular distribution of stiffness and mass in a structure. It becomes less accurate as the structure varies from this assumption. Historically, regular buildings have performed better in earthquakes than have irregular buildings. Layouts prone to damage are: torsionally eccentric ones, "in" and "out" of plane offsets of the lateral system, and buildings with a weak storey (Tremblay and DeVall, 2006). For more details on building configuration issues refer to Chapter 6 of Naeim (2001).

Figure 1-24 illustrates the NBCC 2005 irregularity types. Note that Types 1 to 6 are vertical (elevation) irregularities, while Types 7 and 8 are horizontal (plan) irregularities.

According to NBCC 2005 Clause 4.1.8.7, the structure is considered to be "regular" if it has none of the eight types of irregularity, otherwise it is deemed to be "irregular". The default method of analysis is the dynamic method, but the equivalent static method may be used if <u>any</u> of the following conditions are satisfied:

- the seismic hazard index $I_E F_a S_a(0.2) < 0.35$, or
- the structure is regular, less than 60 m in height, and has a period T < 0.5 seconds in either direction, or
- the structure is irregular, but does not have Type 7 Irregularity, and is less than 20 m in height with period T < 0.5 seconds in either direction.

For single-storey structures such as warehouses and other low-rise masonry buildings, only irregularity Types 7 and 8 might apply, and these would not likely prevent the use of the equivalent static method.

Type 8 irregularity concerns SFRS(s) which are not oriented along a set of orthogonal axes. The structures with this type of irregularity may require more complex seismic analysis in which seismic loads in two orthogonal directions would need to be considered concurrently. According to Clause 4.1.8.8.(1).b), where the components of the SFRS are not oriented along a set of orthogonal axes, and the structure is in a low seismic zone ($I_E F_a S_a(0.2) < 0.35$), then independent analysis about any two orthogonal axes is permitted. However, where the components of the SFRS are not oriented along a set of orthogonal axes, and the structure is in a medium or high seismic zone ($I_E F_a S_a(0.2) \geq 0.35$), then the analysis of the structure can be done independently about any two orthogonal axes for 100% of the prescribed earthquake loads in one direction concurrently with 30% of the prescribed earthquake loads acting in the perpendicular direction (see Clause 4.1.8.8.(1).c). This is so-called "100+30%" rule discussed in Section 1.5.9.3.

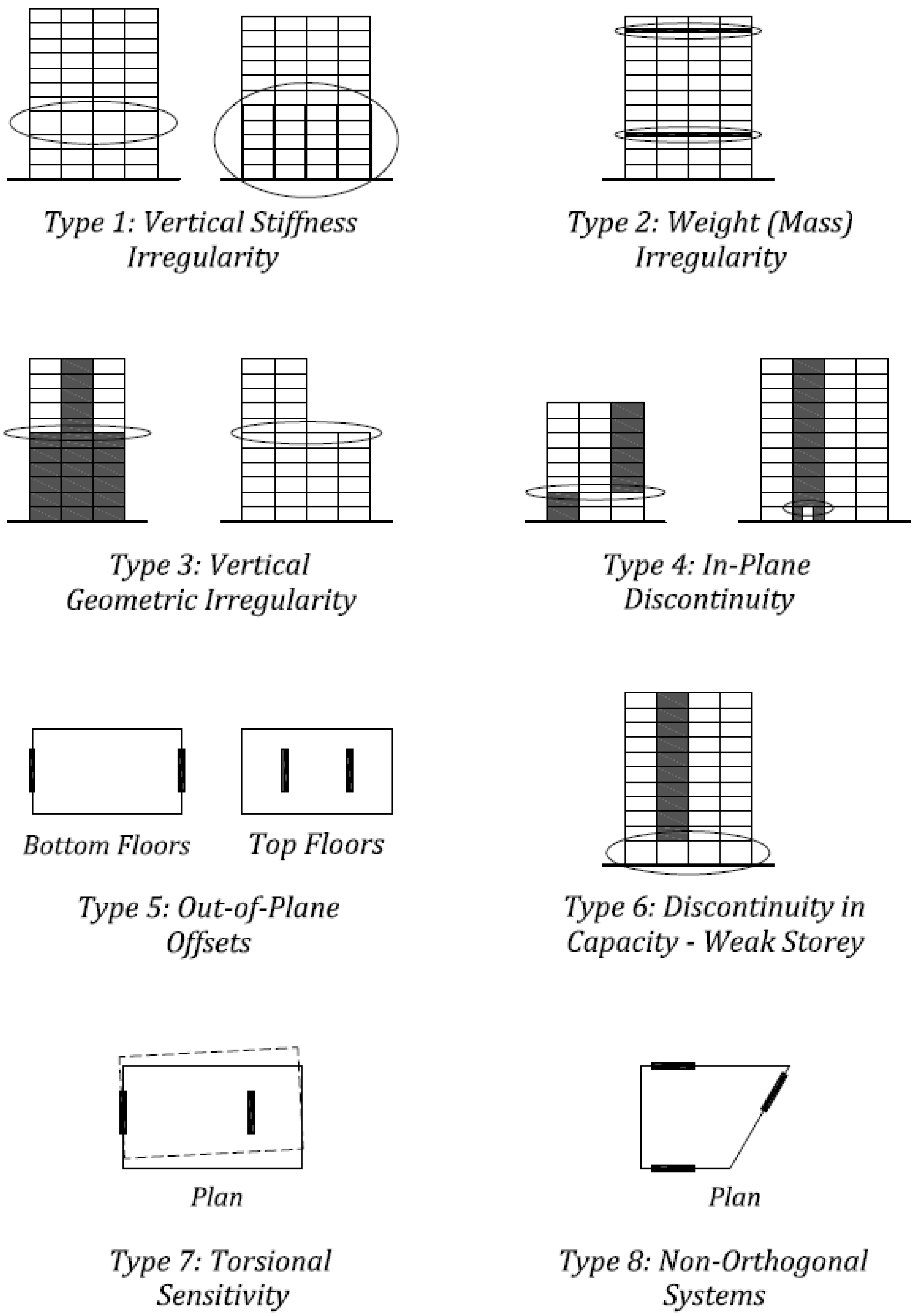


Figure 1-24. Types of irregularity according to NBCC 2005 (Tremblay and DeVall, 2006).

1.5.10.2 Restrictions

4.1.8.10.

Restrictions in NBCC 2005 are based on (i) the natural period or height of the building, (ii) whether the building is in a "high" or "low" seismic zone, (iii) irregularities, and (iv) the importance category of the building. These restrictions are outlined below:

1. Except as required by Clause 4.1.8.10.(2).b), structures with Type 6 irregularity, Discontinuity in Capacity – Weak Storey, are not permitted unless $I_E F_a S_a(0.2) < 0.20$ and the forces used for design of the SFRS are multiplied by $R_d R_o$.
2. Post-disaster buildings shall
   a) not have any irregularities conforming to Types 1, 3, 4, 5, and 7 as described in Table 4.1.8.6, in cases where $I_E F_a S_a(0.2) \geq 0.35$,
   b) not have a Type 6 irregularity as described in Table 4.1.8.6, and
   c) have an SFRS with an $R_d \geq 2.0$.
3. For buildings having fundamental lateral periods $T_a > 1.0s$, and where $I_E F_a S_a(0.2) > 0.25$, walls forming part of the SFRS shall be continuous from their top to the foundation and shall not have irregularities of Type 4 or 5 as described in Table 4.1.8.6.

Note that Table 1-15 in this document is the same as NBCC 2005 Table 4.1.8.6.

Commentary

An important restriction for masonry construction concerns post-disaster structures. In other than low seismic regions the structure cannot have irregularity Types 1, 3, 4, 5, or 7; and must have an $R_d \geq 2.0$. Thus masonry post-disaster structures must be designed with moderately ductile shear walls (with $R_d = 2.0$), and except in low seismic regions (where $I_E F_a S_a(0.2) < 0.35$) the above noted irregularity types should be avoided.

Irregularity Type 6, Discontinuity in Capacity-Weak Storey, is an important restriction for multi-storey structures, and *cannot be present at all in post-disaster structures*. For structures with this type of irregularity, the forces used in the design of the SFRS, except in very low seismic areas, must be multiplied by $R_d R_o$, which implies that the members must remain elastic. This type of irregularity is considered very dangerous as in past earthquakes many structures with weak storeys have had a total collapse of that storey, which has resulted in many deaths. This type of seismic response has often been reported in reinforced concrete frame structures with masonry infill walls which contain more infills in the storeys above the ground floor, leaving the first storey as a weak storey.

1.5.11 Deflections and Drift Limits

4.1.8.13

Lateral displacement (deflection) limits are prescribed in terms of maximum drift. *Drift* means the lateral deflection of one floor (or roof) relative to the floor below. *Drift ratio* is the drift divided by the storey height between the two floors, and is thus a measure of the distortion of the structure.

The NBCC 2005 drift limits are based on the storey height h_s, as follows:

- 0.01 h_s for post-disaster buildings
- 0.02 h_s for schools, and
- 0.025 h_s for all other buildings.

Commentary

Since large deflections and drifts due to earthquakes contribute to (i) damage to the non-structural components, (ii) damage to the elements which are not a part of the SFRS, and (iii) P-Delta effects, NBCC 2005 provisions have moved in the direction of tightening up the drift limits from the previous versions. NBCC 2005 drift limits are more restrictive than those stated in NBCC 1995 because they apply to displacements based on a 1/2475 year return period event, whereas the NBCC 1995 uses the 1/475 year event (DeVall, 2003). Specifically, tighter drift limits for post-disaster or school buildings reflect the importance of these structures.

Drift and drift ratio can be explained on an example of a three-storey building shown in Figure 1-25. The drift in say the second storey is equal to $\Delta_2 - \Delta_1$, where Δ_1 and Δ_2 denote lateral deflections at the first and second floor level respectively. The corresponding drift ratio for that storey is equal to $(\Delta_2 - \Delta_1)/h$, where $h = h_2 - h_1$ (storey height). The average drift ratio for the entire structure is $(\Delta_3)/h$.

Drifts are the elastic deflections and need not be increased by the importance factor I_E as that has already been accounted for in the drift limits. If the equivalent static forces, which are the elastic forces multiplied by $I_E/R_d R_o$, are applied to the elastic structure to calculate deflections, then these deflections must be multiplied by $R_d R_o/I_E$ to get realistic values of the deflections.

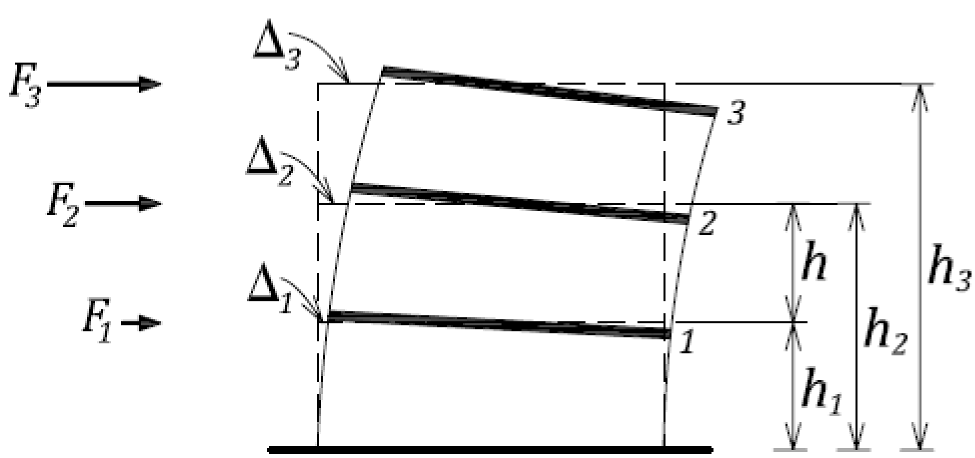


Figure 1-25. Lateral deflections and drift.

In checking drift limits the drift should be taken at the location on the floor which has the maximum deflection. Torsional effects can result in corner deflections being much larger than the deflection at the centre of the floor plan.

Since deflections increase with an increase in the period T , the stiffness used in calculating the deflections should reflect a softening of the structure (before yielding occurs) that might come from cracking of the masonry. The stiffness for squat shear walls should be determined taking into account shear deformation. If the period T determined per NBCC provisions (see Section

1.5.4) is used to determine the seismic forces, the stiffness of the structure used in calculating the deflections should be such that the calculated period would not be less than the NBCC period. Many masonry structures are very stiff and the deflections will be well below the code limits, and so displacement calculations will not be critical in many cases.

Drift limits are imposed so that members of the SFRS will not be subjected to large lateral displacements that might degrade their ability to resist the seismic loads, but also to ensure that members that are not part of the SFRS, such as columns that support gravity load only, should not fail during the earthquake. The seismic portion of the code is mute on drift limits for serviceability, however the designer can estimate the structural deflections at different hazard levels, since displacements are roughly proportional to the level of hazard. For example, the drift at an exceedance probability of 1/475 per annum would be about half of that for the 1/2475 per annum design drift because the 1/475 per annum hazard is roughly half the 1/2475 per annum hazard.

1.5.12 Dynamic Analysis Method

4.1.8.12

In NBCC 2005 the default analysis method is the dynamic method. For many structures, even though the equivalent static analysis method could be used according to NBCC seismic provisions, dynamic analysis may be used for other reasons. The purpose of this section is not to explain how to use dynamic analysis software, but to give some guidance on scaling or comparing the dynamic results with the results from the static method.

The base shear from a dynamic analysis, determined using the site design spectrum, will give the dynamic elastic base shear, V_e. NBCC 2005 requires that for regular buildings if the base shear from the dynamic method is less than 0.8 times the base shear from the static method, then the dynamic results should be scaled to give 0.8 of the static base shear. If the structure is deemed to be irregular, then the dynamic results should be scaled to 100% of the static results. In essence this means that the dynamic results cannot be less than the static results (or 80% of the static results for regular structures), but if they are larger they should not be reduced to the static values. The comparison can be made on the basis of the elastic base shear, V_e, or the design base shear, V, but must be the same for both analyses.

Since the static analysis method is allowed to reduce the design base shear by a factor of two-thirds in the short period range while the dynamic analysis method must use the design spectrum *S(T)*, it is very unlikely that for short period structures the base shear determined using the dynamic method would ever be less than that given by the static method, let alone less than the 80% value allowed for regular buildings. This is an inconsistency in the code as it adversely impacts the results from a dynamic analysis for short period structures, but not for longer period structures. It is anticipated that the NBCC code, or at least the commentary to the code, will be changed for the next edition due out in 2010, allowing the base shear from a dynamic analysis be evaluated using a spectrum where the short period values are reduced by one third.

If the building is very eccentric, a 3-D dynamic analysis will produce a low total base shear. In that case, it would be very conservative to require that these low base shears be scaled to the static base shear, since the static method of determining the base shear *V* does not consider torsional motion. To make a fair comparison between the static and dynamic results the suggestion is to perform a dynamic analysis with the rotation of the structure restrained about a vertical axis, and then compare the resulting base shear to the static value to determine the amount of scaling required, if any.

Scaling, if necessary, should be applied to the member forces determined from the full 3-D dynamic analysis multiplied by $I_E / R_d\ R_o$ to give the design member forces. The design displacements are the elastic displacements given by the dynamic analysis, and scaled if necessary. To these design forces and displacements must be added the forces and displacements from accidental torsion.

1.5.13 Soil-Structure Interaction

For large structures located on soft soil sites the deformation of the soil may have an appreciable influence on the response of the structure. The most common type of soil-structure interaction is based on the flexibility of the soil, which is usually represented by a lateral spring between the foundation and the point where the seismic motion is input, and with a rotational spring at the base of flexural walls. There is a second type of soil-structure interaction, termed the kinematic interaction, which only applies to structures with a very large plan area or a deep foundation, and which will not be discussed further here.

The effect of introducing springs between the point of input motion and the foundation is to increase the period of the structure, which usually reduces the seismic forces but increases the deflections. In the case of a wall structure, the increased deflections may not increase the deformation of the wall since they would arise from rotations of the foundation, but they would increase the interstorey drifts which would have an influence on other parts of the structure. While it is not so apparent, the larger deflections may lead to larger inelastic deformations and larger ductility demands. However, at the small ductilities used in masonry design this is most likely not to be a concern.

For masonry structures, soil-structure interaction will likely only have an influence for slender wall structures with individual footings, where rotation of the footing would have a large effect on the wall displacement. The determination of the soil stiffness should be left to an experienced geotechnical engineer, but it should be recognized that the precision at which the soil stiffness can be estimated is quite low. It is common to consider quite wide upper and lower bounds on the estimated stiffness of the soil springs.

TABLE OF CONTENTS – CHAPTER 2

2 SEISMIC DESIGN OF MASONRY WALLS TO CSA S304.1-04

2.1 Introduction

Chapter 1 provides background on the seismic response of structures and seismic analysis methods, and explains key NBCC 2005 seismic provisions relevant to masonry design. This chapter provides an overview of seismic design requirements for reinforced masonry walls. Relevant CSA S304.1-04 design requirements are presented, along with related commentary, to provide detailed explanations of the NBCC provisions. Topics range from reinforced masonry shear walls subjected to in-plane and out-of-plane seismic loads, to a number of special topics such as masonry infill walls, stack pattern walls, veneers, and construction-related issues. Differences between CSA S304.1-04 seismic design requirements and those of the previous (1994) edition are identified and discussed, along with their design implications. For easy reference, relevant CSA S304.1 clauses are shown in a framed textbox where appropriate. Appendix B contains research findings and international code provisions related to seismic design of masonry structures. Appendix C contains relevant design background used in the design examples included in Chapter 4.

2.2 Masonry Walls – Basic Concepts

Structural walls are the key structural components in a masonry building and are used to resist some or all of the following load effects:

- axial compression due to vertical gravity loads,
- out-of-plane bending (flexure) and shear due to transverse wind, earthquake or blast loads and/or eccentric vertical loads, and
- in-plane bending and shear due to lateral wind and earthquake loads applied to building system in a direction parallel to the plane of the wall.

In a masonry building subjected to earthquake loads, horizontal seismic inertia forces develop in the walls, and the floor and roof slabs. The floor and roof slabs are called diaphragms where they transfer lateral loads to the lateral load resisting system. These inertia forces are proportional to the mass of these structural components and the acceleration at their level. An isometric view of a simple single-storey masonry building is shown in Figure 2-1a (note that roof diaphragm has been omitted for clarity). For earthquake ground motion acting in the direction shown in the figure, the roof diaphragm acts like a horizontal beam spanning between walls A and B. The end reactions of this beam are transferred to the walls A and B. These walls, subjected to lateral load along their longitudinal axis (also called *in-plane* loads), are called *shear walls*. Along with the floor and roof diaphragms, shear walls are the components of the building's lateral load path that transfers the lateral load to the foundations. A well-designed and well-built masonry building has a reliable load path, established by design, which transfers the forces over the full height of the building from the roof to the foundation.

Note also that the earthquake ground motion causes vibration of the transverse walls C and D. These walls are subjected to inertia forces proportional to their self-weight and are loaded *out-of-plane* (or transverse to their longitudinal axis). A vertical section through wall D that is loaded in the out-of-plane direction is shown in Figure 2-1b, while an elevation of shear wall A and its in-plane loading is shown in Figure 2-1c.

It is important to note that walls are subjected to shear forces in both the in-plane and out-of-plane directions during an earthquake event. However, the main difference between *shear walls* and other types of walls is that shear walls are key vertical components of a lateral load resisting system for a building, referred to as the Seismic Force Resisting System or SFRS by NBCC 2005. Usually not all walls in the building are shear walls; some walls (loadbearing and/or nonloadbearing) are not intended to resist in-plane loads, and are not designed and detailed as shear walls; in that case, they cannot be considered to form a part of the SFRS.

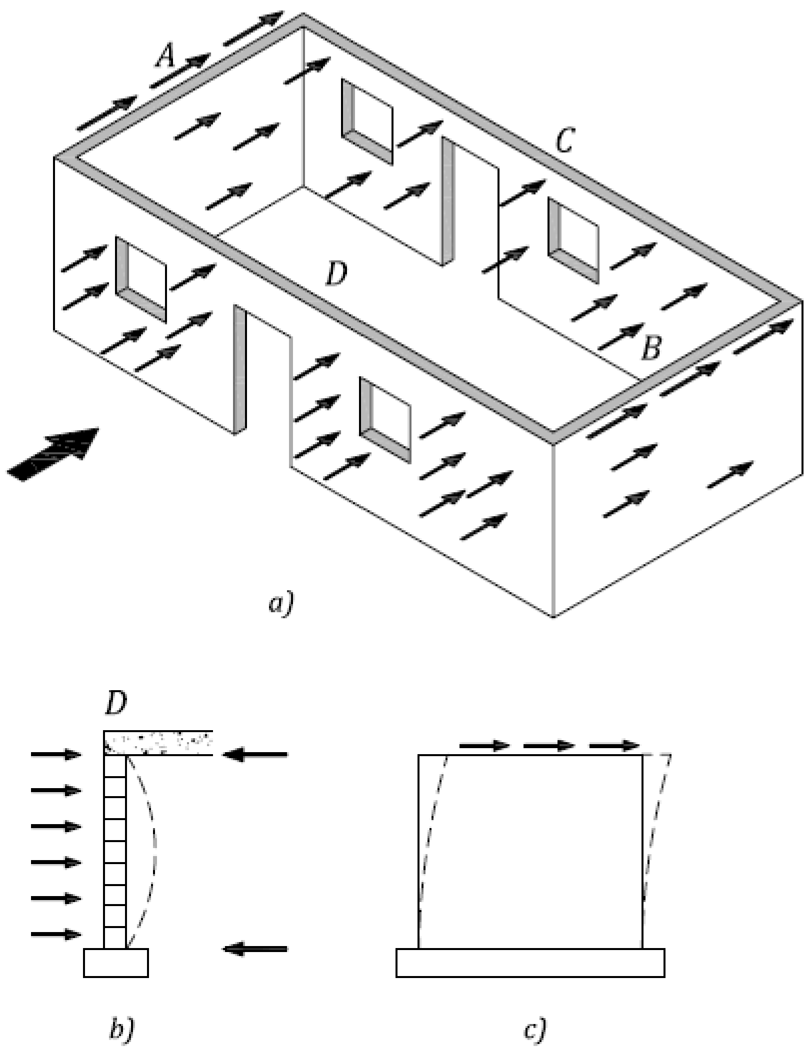


Figure 2-1 Simple masonry building: a) isometric view showing lateral loads; b) out-of-plane loads; c) in-plane loads (resisted by shear walls).

A typical reinforced concrete block masonry wall is shown in Figure 2-2. Vertical reinforcing bars are placed in the open cells of the masonry units (note that the term *cores* is also used in masonry construction practice), and are provided at generally uniform spacing along the wall length. The role of vertical reinforcement is to enhance the ability of the wall to resist forces due to vertical loads, and forces resulting from induced moments due to vertical eccentricities, as well as the out-of-plane loads. Horizontal wall reinforcement is usually provided in two forms: i) ladder- or truss-type wire reinforcement placed in mortared bed joints (see Figure 2-2b), and ii) steel bars (similar to vertical reinforcement) placed in grouted bond beams at specified locations along the wall height (see Figure 2-2c). Horizontal wire and bar reinforcement restrict in-plane movements due to temperature and moisture changes, resist in-plane shear forces and/or forces due to moments caused by out-of-plane loads. Grout, similar to concrete but with higher slump, is used to fill the cells of the masonry units that contain vertical and horizontal reinforcement bars. Grout increases the loadbearing capacity of the masonry by increasing its area, and serves to bond the reinforcement to the masonry unit so that the reinforcement and unit act compositely.

Grade 400 steel (yield strength 400 MPa) is nearly always used for horizontal and vertical reinforcing bars, whereas cold-drawn galvanized wire is used for joint reinforcement (also known as American Standard Wire Gauge – ASWG). Yield strength for joint reinforcement varies, but it usually exceeds 480 MPa for G30.3 steel wire. In design practice, 400 MPa yield strength is used both for the reinforcement bars and the joint wire reinforcement. The properties of concrete masonry units are summarized in Appendix D, while mechanical properties of masonry and steel materials are discussed by Drysdale and Hamid (2005) and Hatzinikolas and Korany (2005). The material resistance factors for masonry and steel prescribed by CSA S304.1-04 are as follows:

ϕ_m = 0.6 resistance factor for masonry (Cl.4.3.2.1)

ϕ_s = 0.85 resistance factor for steel reinforcement (Cl.4.3.2.2)

The following notation will be used to refer to wall dimensions (see Figure 2-2a):

l_w - wall length

h_w - total wall height

t - overall wall thickness

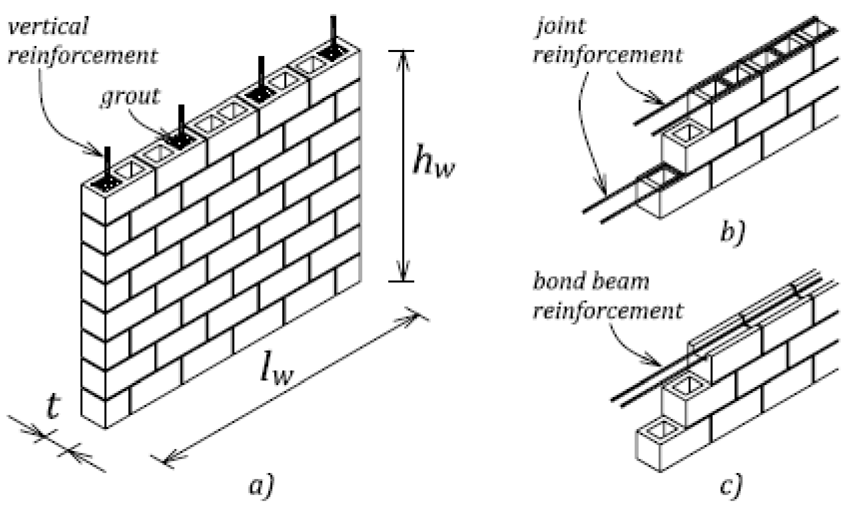


Figure 2-2. Typical reinforced concrete masonry block wall: a) vertical reinforcement; b) joint reinforcement; c) bond beam reinforcement.

Typical reinforced concrete masonry wall construction is shown in Figure 2-3. The lower section of the wall has been grouted to the height of a bond beam course. Vertical bars extend above the bond beam to serve as bar splices for the continuous vertical reinforcement placed in the next wall section.

Figure 2-3 Masonry wall under construction (Credit: Masonry Institute of BC).

Walls in which only the reinforced cells are grouted are called *partially grouted walls*, whereas the walls in which all the cells are grouted are called *fully grouted walls*. Irrespective of the extent of grouting (partial/full grouting), cross-sectional area of the entire wall section (considering the overall thickness t) is termed *gross cross-sectional area*, A_g. In partially grouted or hollow (ungrouted) walls, the term *effective cross-sectional area,* A_e, denotes that area which includes the mortar-bedded area and the area of grouted cells (S304.1 Cl.10.3). Both the gross and effective wall area are shown for a wall strip of unit length (usually equal to 1 metre). The difference between A_g and A_e is illustrated in Figure 2-4. In ungrouted masonry construction, the webs are generally not mortared, however in partially grouted reinforced masonry construction, the webs on each side of a grouted cell are sometimes mortared to ensure that grout does not flow into the adjacent cells not intended to be grouted. In any case, coarse grout will flow from the grouted core to fill the gap between the webs adjacent to the cell. In exterior walls the effective area is often significantly reduced by raked joints (this is not a concern with a standard concave tool joint). The designer should consider this effect in the calculation of the depth of the compression stress block.

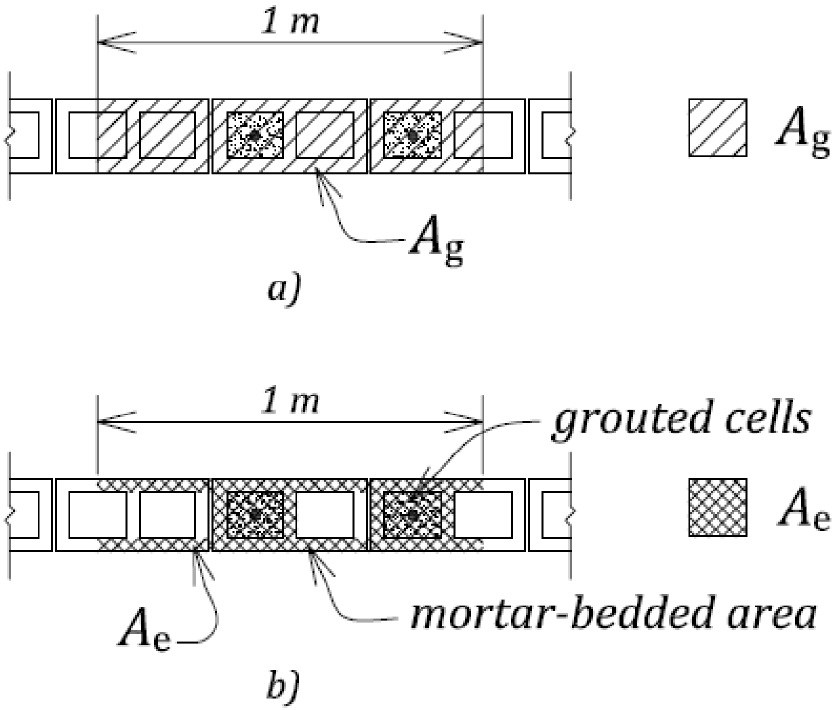


Figure 2-4. Wall cross-sectional area: a) gross area; b) effective area.

Shear walls without openings (doors and/or windows) are referred to as *solid* walls (Figure 2-5a), while the walls with door and/or window openings are referred to as *perforated* walls (Figure 2-5b). Regions between the openings in a perforated wall are called *piers* (see piers A, B, and C in Figure 2-5b). Perforated shear walls in medium-rise masonry buildings with uniform distribution of vertically aligned openings over the wall height are called *coupled walls.*

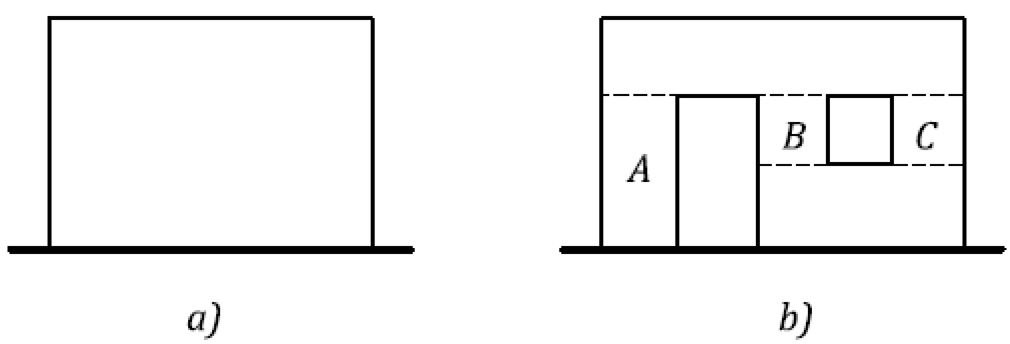


Figure 2-5. Masonry shear walls: a) solid, and b) perforated.

Depending on the wall geometry, in particular the height/length (h_w/l_w) aspect ratio, shear walls are classified into the following two categories:

- Flexural shear walls with a height/length aspect ratio of 1.0 or higher (Figure 2-6a), and
- Squat shear walls with a height/length aspect ratio less than 1.0 shown in Figure 2-6b (see S304.1 Cl.4.6.6 and 10.10.1.3).

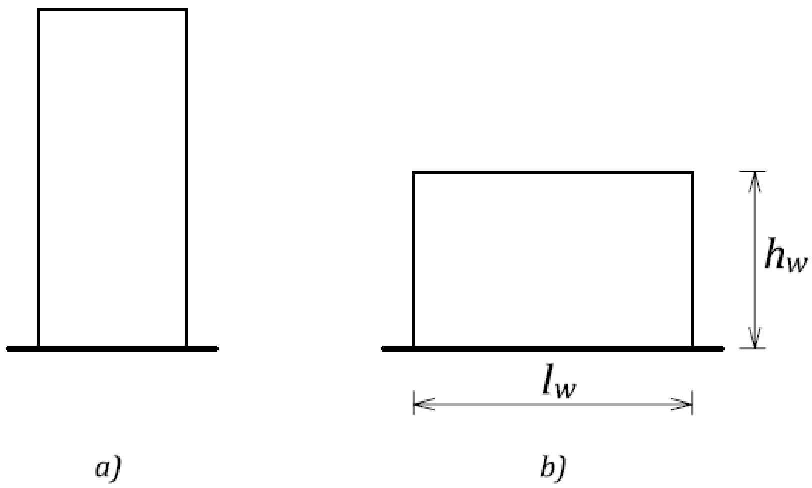
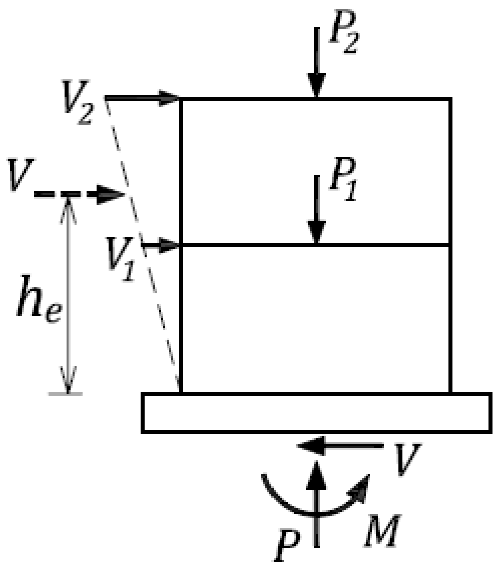


Figure 2-6. Shear wall classification based on the aspect ratio: a) flexural walls; b) squat walls.

Depending on whether the walls resist the effects of gravity loads in addition to other loads, masonry walls can be classified as loadbearing or nonloadbearing walls. *Loadbearing* walls resist the effects of superimposed gravity loads (in addition to their selfweight) plus the effects of lateral loads. *Nonloadbearing* walls resist only the effects of their selfweight and possibly out-of-plane wind and earthquake loads. Shear walls are loadbearing walls, irrespective of whether they carry gravity loads or not.

In masonry design, selection of the locations where movement joints (also known as control joints) need to be provided is one of the initial and very important detailing decisions. Some movement joints are provided to facilitate design and construction while others prevent cracking at undesirable locations. In any case, wall length is determined by the location of movement joints and so this detailing decision carries an implication for seismic design. For more details on movement joints refer to MIBC (2008).

In general, shear walls are subjected to lateral loads at the floor and roof levels, as shown in Figure 2-7. (Note the inverse triangular distribution of lateral loads simulating earthquake effects.) Distribution of forces in a shear wall is similar to that of a vertical cantilevered beam fixed at the base. Figure 2-7 also shows internal reactive forces acting at the base of the wall. Note that the wall section at the base is subjected to the shear force, V , equal to the sum of the horizontal forces acting on the wall, the bending moment, M , due to all horizontal forces acting at the effective height h_e , and the axial force, P , equal to the sum of the axial loads acting on the wall.



$$P = \sum P_i$$
$$V = \sum V_i$$
$$M = V \cdot h_e$$

Figure 2-7. Load distribution in shear walls.

2.3 Reinforced Masonry Shear Walls Under In-Plane Seismic Loading

2.3.1 Behaviour and Failure Mechanisms

The behaviour of a reinforced masonry shear wall subjected to the combined effect of horizontal shear force, axial load and bending moment depends on several factors. These include the level of axial compression stress, the amount of horizontal and vertical reinforcement, the wall aspect ratio, and the mechanical properties of the masonry and steel. The two main failure mechanisms for reinforced masonry shear walls are:

- Flexural failure (including ductile flexural failure, lap splice slip, and flexure/out-of-plane stability), and
- Shear failure (includes diagonal tension failure and sliding shear failure).

Each of these failure mechanisms will be briefly described in this section. The focus is on the behaviour of walls subjected to cyclic lateral load simulating earthquake effects. Failure mechanisms for reinforced masonry walls are discussed in detail in FEMA 306 (1999).

2.3.1.1 Flexural failure mechanisms

Ductile flexural failure is found in reinforced walls and piers characterized by a height/length aspect ratio (h_w/l_w) of 1.0 or higher and a moderate level of axial stress (less than $0.1f'_m$). This failure mode is characterized by tensile yielding of vertical reinforcement at one end of the wall and simultaneous cracking and possible spalling of masonry units and grout in the toe area (compression zone). In some cases, buckling of compression reinforcement accompanies the cracking and spalling of masonry units. Experimental studies have shown that the vertical reinforcement is effective in resisting tensile stresses and that it yields shortly after the cracking in masonry takes place (Tomazevic, 1999). Damage is likely to include both horizontal flexural cracks and diagonal shear cracks of small size concentrated in the plastic hinge zone, as shown in Figure 2-8a. (The plastic hinge zone is the region of the member where inelastic deformations occur and it will be discussed in Section 2.5.4.2.) In general, this is the preferred failure mode for reinforced masonry shear walls, since the failure mechanism is ductile and effective in dissipating earthquake-induced energy once the yielding of vertical reinforcement takes place.

Flexure/lap splice slip failure may take place when starter reinforcing bars projecting from the foundations have insufficient lap splice length, or when the rebar size is large relative to wall thickness (e.g. 25M bars used in 200 mm walls), resulting in bond degradation and eventual rocking of the wall at the foundation level. Initially, vertical cracks appear at the location of lap splices followed by the cracking and spalling at the toes of the wall (see Figure 2-8b). This mode of failure may be fairly ductile but it results in severe strength degradation and does not provide much energy dissipation.

Flexure/out-of-plane instability takes place at high ductility levels (see Figure 2-8c). Ductility is a measure of the capacity of a structure to undergo deformation beyond yield level while maintaining most of its load-carrying capacity (ductile seismic response will be discussed in Section 2.5.3). When large tensile strains develop in the tensile zone of the wall, that zone can become unstable when the load direction reverses in the next cycle and the compression takes place. This type of failure has been observed in laboratory tests of well detailed highly ductile flexural walls (Paulay and Priestley, 1993), but it has not been observed in any of the post-earthquake field surveys so far (FEMA 306, 1999). This failure mechanism can be prevented by ensuring stability of the wall compression zone through seismic design (see Section 2.5.4.4 for more details).

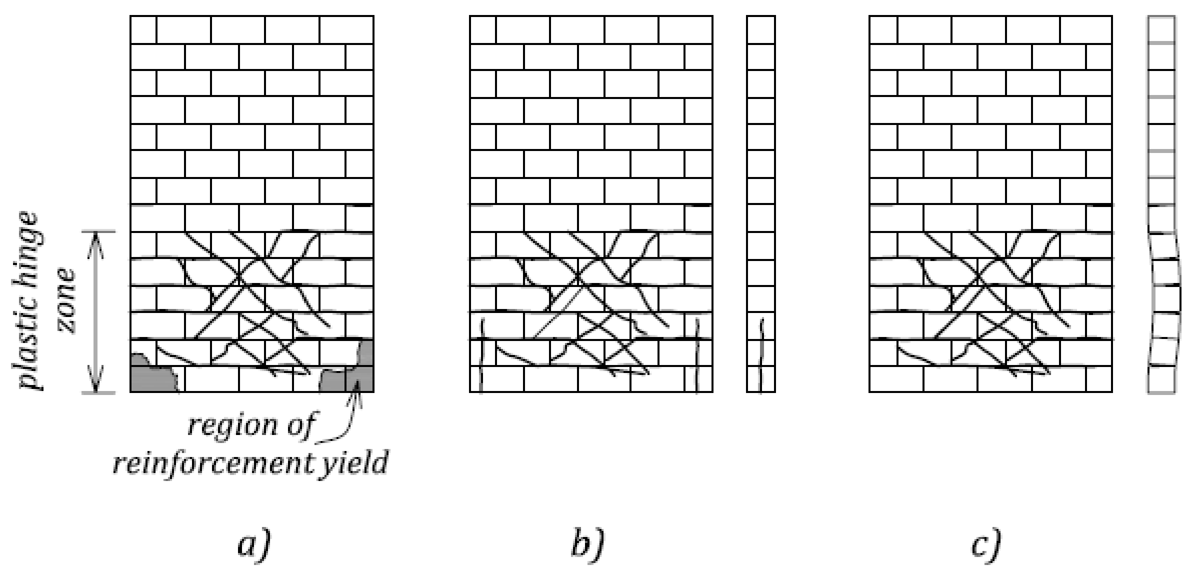


Figure 2-8. Flexural failure mechanisms: a) ductile flexural failure; b) lap splice slip, and c) out-of-plane instability (FEMA 306, 1999, reproduced by permission of the Federal Emergency Management Agency).

2.3.1.2 Shear failure mechanisms

Shear failure is common in masonry walls subjected to seismic loads and has been observed in many post-earthquake field surveys. Due to the dominant presence of diagonal cracks, this mode is also known as *diagonal tension* failure (see Figure 2-9a). It usually takes place in walls and piers characterized by low aspect ratio (h_w/l_w less than 0.8). These walls are usually lightly reinforced with horizontal shear reinforcement and so the shear failure takes place before the wall reaches its full flexural capacity.

This mode of failure is initiated when the principal tensile stresses due to combined horizontal seismic loads and vertical gravity loads exceed the masonry tensile stress. When the amount and anchorage of horizontal reinforcement are not adequate to transfer the tensile forces across the first set of diagonal cracks, the cracks continue to widen and result in a major X-shaped diagonal crack pair, thus leading to a relatively sudden and brittle failure. Note that these "diagonal cracks" may develop either trough the blocks, or along the mortar joints.

In modern masonry construction designed according to the code requirements, it is expected that adequate horizontal reinforcement is provided and that it is properly anchored within wall end zones. Horizontal reinforcement can be effective in resisting tensile forces in the cracked wall and in enhancing its load-carrying capacity. After the initial diagonal cracks have been formed, several uniformly distributed cracks develop and gradually spread in the wall. Failure occurs gradually as the strength of the masonry wall deteriorates under the cyclic loading. Voon (2007) refers to this mechanism as "ductile shear failure". It should be noted that ductile behaviour is usually associated with the flexural failure mechanism, while shear failure mechanisms are usually characterized as brittle, however in very squat shear walls a ductile shear mechanism may be the only ductile alternative.

Sliding shear failure may take place in masonry walls subjected to low gravity loads and rather high seismic shear forces. This condition can be found at the base level in low-rise buildings or at upper storeys in medium-rise buildings where accelerations induced by earthquake ground motion are high, but it can also take place at other locations. Sliding shear failure takes place when the shear force across a horizontal plane (usually base in reinforced masonry walls) exceeds the frictional resistance of masonry and a horizontal crack is formed at the base of the wall, as shown in Figure 2-9b. There may be very limited cracking or damage in the wall outside the sliding joint. The frictional mechanism at the sliding interface is activated after the clamping force develops in vertical reinforcement which yields in tension. Even though this mode of failure is often referred to as shear failure mode, it may also take place in the walls characterized by flexural behaviour. Pre-emptive sliding at the base limits the development of full flexural capacity in the wall.

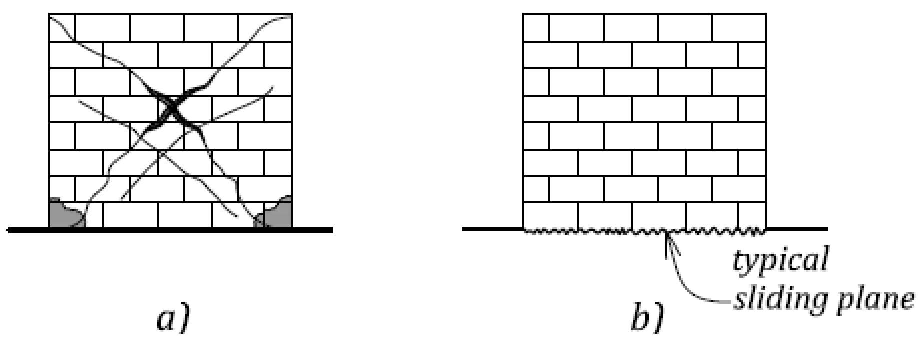


Figure 2-9. Shear failure mechanisms: a) diagonal tension[1], and b) sliding shear.

2.3.2 Shear/Diagonal Tension Resistance

Shear resistance of reinforced masonry shear walls depends on several parameters, including the masonry compressive strength, grouting pattern, amount and distribution of horizontal reinforcement, magnitude of axial stress, and height/length aspect ratio. Over the last two decades, significant experimental research studies have been conducted in several countries, including the US, Japan, and New Zealand. Although the findings of these studies have confirmed the influence of the above parameters on the shear resistance of masonry walls, it appears to be difficult to quantify the influence of each individual parameter. This is because of

[1] Source: FEMA 306, 1999, reproduced by permission of the Federal Emergency Management Agency

the complexity of shear resistance mechanisms and a lack of effective theoretical models. As a result, shear resistance equations included in the Canadian masonry design standard, S304.1-04, and those of other countries are based on statistical analyses of test data obtained from a variety of experimental studies. The diagonal tension shear resistance equation for reinforced masonry walls in CSA S304.1 (both 1994 and 2004 editions) is mainly based on the research by Anderson and Priestley (1992) and other research based on wall tests in the US and Japan. Refer to Section B.1 for a detailed research background on the subject.

This section discusses the in-plane shear resistance provisions of CSA S304.1-04 for non-seismic conditions; seismic requirements related to shear design are discussed in Section 2.5.4.5. The design of walls built using running bond is discussed in this section, and walls built using the stack pattern are discussed in Section 2.6.3.

2.3.2.1 Flexural shear walls

10.10.1.1

Flexural shear walls are characterized by height/length aspect ratio of 1.0 or higher (see Figure 2-6a). Consider a reinforced masonry shear wall built in running bond which is subjected to the effect of factored shear force, V_f, and the factored bending moment, M_f.

Factored in-plane shear resistance, V_r, is determined as the sum of contributions from masonry, V_m, and steel, V_s, that is,

$$V_r = V_m + V_s \quad (1)$$

Masonry shear resistance, V_m, is equal to:

$$V_m = \phi_m (v_m b_w d_v + 0.25 P_d) \gamma_g \quad (2)$$

Wall dimensions (b_w and d_v):
$b_w = t$ overall wall thickness (mm) (referred to as "web width" in CSA S304.1); note that b_w does not include flanges in the intersection walls
d_v = effective wall depth (mm)
$d_v \geq 0.8 l_w$ for walls with flexural reinforcement distributed along the length
Wall cross-sectional dimensions (b_w and d_v) used for shear design calculations are illustrated in Figure 2-10.

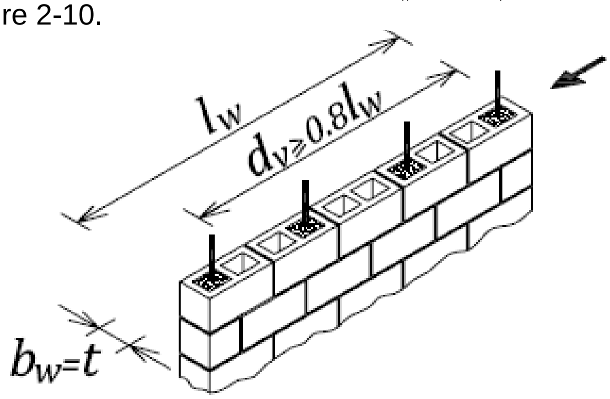


Figure 2-10. Wall cross-sectional dimensions used for in-plane shear design.

Effect of axial load (P_d):

P_d = axial compression load on the section under consideration, based on 0.9 times dead load, P_{DL}, plus any axial load, N , arising from bending in coupling beams or piers (see Figure 2-11)

$$P_d = 0.9P_{DL} \text{ for solid walls}$$

$$P_d = 0.9P_{DL} \pm N \text{ for perforated/coupled walls}$$

Note that the net effect of tension and compression forces $\pm N$ on the total shear in the wall is equal to 0.

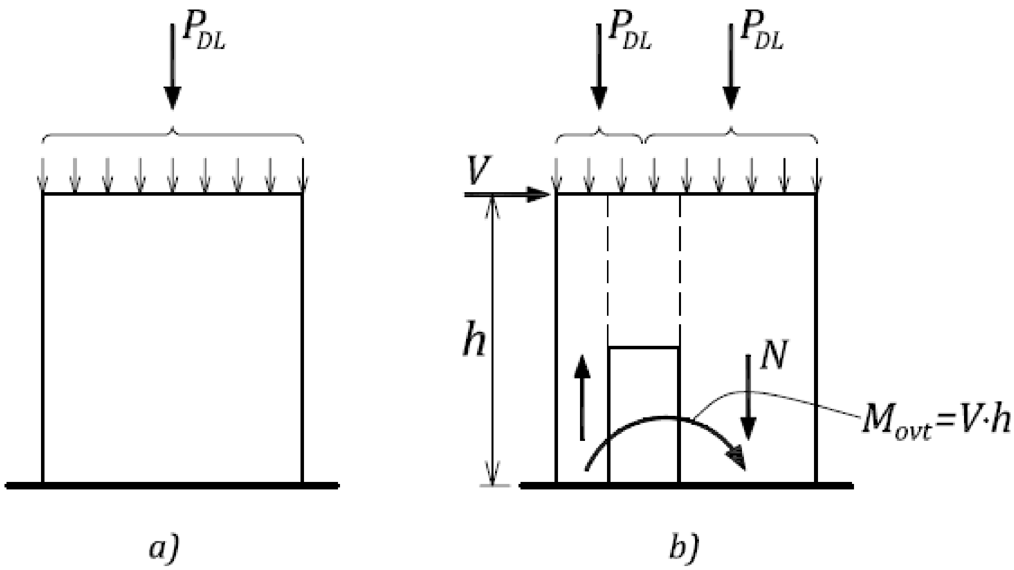


Figure 2-11. Axial load in masonry walls: a) solid; b) perforated.

Effect of grouting (γ_g):

γ_g = factor to account for partially grouted walls that are constructed of hollow or semi-solid units

$$\gamma_g = 1.0 \text{ for fully grouted masonry, solid concrete block masonry, or solid brick masonry}$$

$$\gamma_g = \frac{A_e}{A_g} \text{ for partially grouted walls, but } \gamma_g \le 0.5$$

where (see Figure 2-4)

A_e = effective cross-sectional area of the wall (mm^2)

A_g = gross cross-sectional area of the wall (mm^2)

Masonry shear strength (v_m):

v_m represents shear strength attributed to the masonry in running bond, which is determined according to the following equation:

10.10.1.4

$$v_m = 0.16(2 - \frac{M_f}{V_f d_v})\sqrt{f'_m} \qquad \textit{MPa units} \qquad (3)$$

Shear span ratio ($\frac{M_f}{V_f d_v}$):

The following limits apply to the shear span ratio:

$$0.25 \leq \frac{M_f}{V_f d_v} \leq 1.0$$

10.10.1.1

Reinforcement shear resistance, V_s, is equal to:

$$V_s = 0.6\phi_s A_v f_y \frac{d_v}{s} \quad (4)$$

where
A_v = area of horizontal wall reinforcement (mm^2)
s = vertical spacing of horizontal reinforcement (mm)

As discussed in this section, the factored in-plane shear resistance, V_r, is determined as the sum of contributions from masonry, V_m, and reinforcement, V_s, that is,

$$V_r = V_m + V_s \quad (5)$$

where

$$V_m = \phi_m (v_m b_w d_v + 0.25 P_d)\gamma_g \quad (6)$$

and

$$V_s = 0.6\phi_s A_v f_y \frac{d_v}{s} \quad (7)$$

CSA S304.1 prescribes the following upper limit for the factored in-plane shear resistance V_r for flexural walls:

$$V_r \leq \max V_r = 0.4\phi_m \sqrt{f'_m} b_w d_v \gamma_g \quad (8)$$

Commentary

Axial compression:
The equation for the factored shear resistance of masonry, V_m, in accordance with CSA S304.1 [equation (2)], takes into account the positive influence of axial compression. The term $0.25P_d$ was established based on the statistical analyses of experimental test data carried out by Anderson and Priestley (1992). The 0.25 factor is consistent with that used for concrete in estimating the shear strength of columns.

Consider a masonry shear wall subjected to the combined effect of axial and shear forces shown in Figure 2-12a. A two-dimensional state of stress develops in the wall: axial load, P, causes the axial compression stress, σ, while the shear force, V, causes the shear stress, v. The presence of axial compression stress delays the onset of cracking in the wall since it reduces the principal tensile stress due to the combined shear and compression. Shear cracks develop in the wall once the principal tensile stress reaches the masonry tensile strength (which is rather low). It should be noted, however, that the masonry shear resistance decreases in a wall section subjected to high axial compression stresses (see the diagram shown in Figure 2-12b). This is based on experimental studies – for more details refer to Drysdale and Hamid

(2005). Note that shear walls in low-rise masonry buildings are subjected to low axial compression stresses, as shown in Figure 2-12b.

<u>Grouting pattern:</u>
CSA S304.1-04 takes into account the effect of grouting on the masonry shear resistance through the γ_g factor, which assumes the value of 1.0 for fully grouted walls and 0.5 or less for partially grouted walls. Research evidence indicates that fully grouted reinforced masonry walls demonstrate higher ductility and strength under cyclic lateral loads than otherwise similar partially grouted specimens, as discussed in Section B.5.

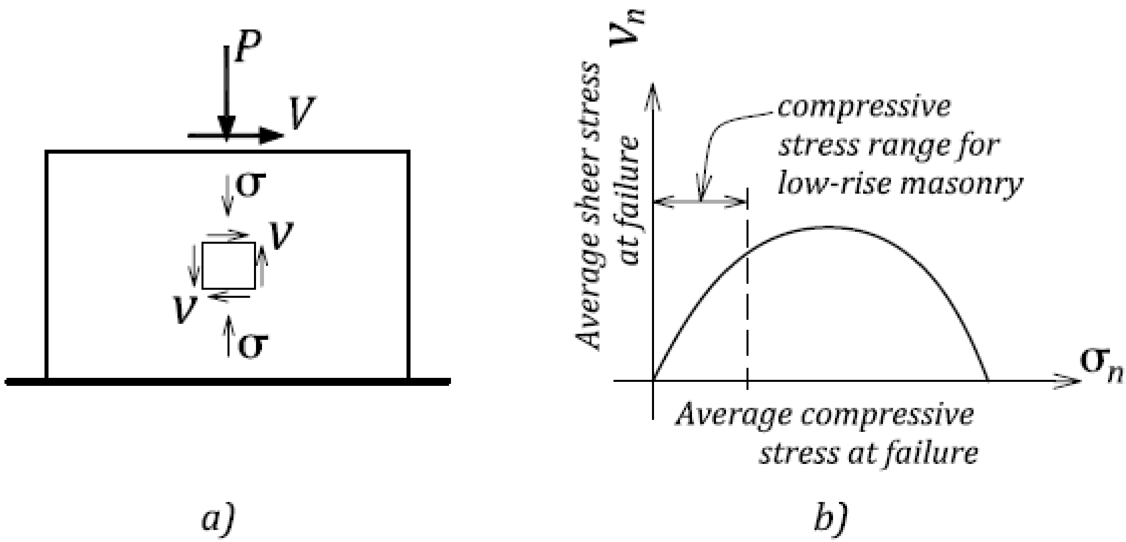


Figure 2-12. Effect of axial stress: a) a shear wall subjected to the combined shear and axial load; b) relationship between the shear stress at failure and the compression stress.

<u>Masonry shear strength (v_m):</u>
Masonry shear strength defined by equation (3) depends on masonry tensile strength represented by the $\sqrt{f'_m}$ term, as well as on the shear span ratio, $M_f / V_f d_v$. Walls with shear span ratios of less than 1.0 behave like squat walls and are characterized by the highest masonry shear resistance, as illustrated in Figure 2-13.

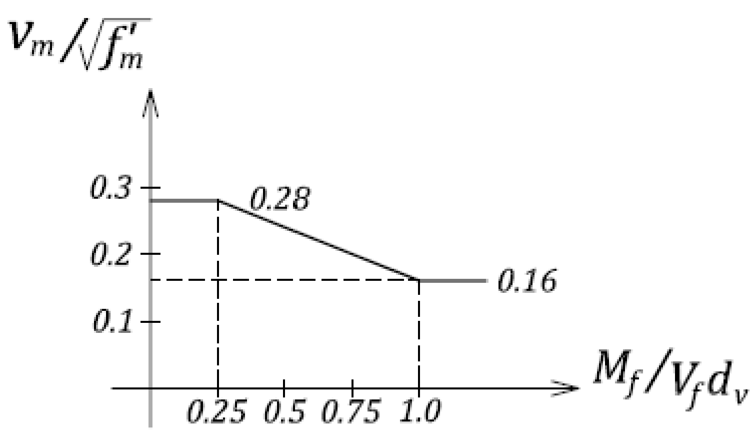


Figure 2-13. Effect of shear span ratio on the masonry shear strength.

For shear walls, the ratio M_f/V_f is equal to the effective height, h_e, at which the resultant shear force V_f acts, thereby causing the overturning moment $M_f = V_f \times h_e$ (see Figure 2-14). The term d_v denotes the effective wall depth, which is equal to a fraction of the wall length, l_w. Hence, $M_f/V_f d_v$ is equal to shear span ratio, h_e/d_v, which is related to the height-to-length aspect ratio.

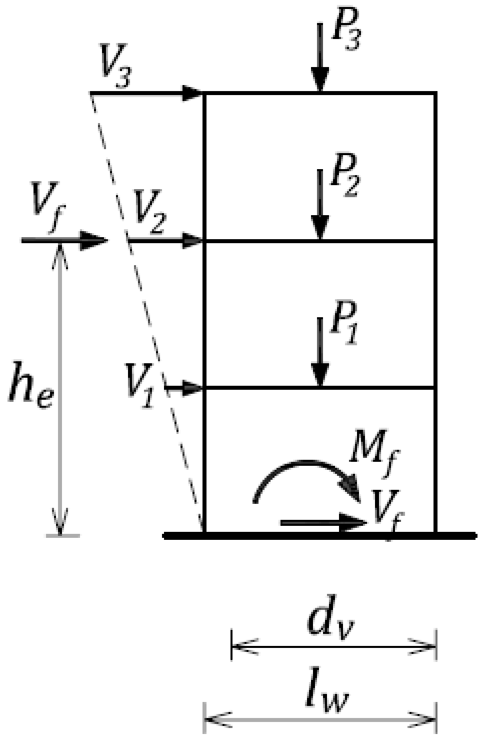


Figure 2-14. Shear span ratio $\frac{h_e}{d_v}$.

<u>Reinforcement shear resistance</u> (V_s):
Reinforcement shear resistance in reinforced masonry shear walls of running bond is mainly provided by horizontal steel bars and/or joint reinforcement. This model assumes that a hypothetical failure plane is at a 45° angle with regard to the horizontal axis, as shown in Figure 2-15a. When diagonal cracking occurs, tension develops in the reinforcing steel crossing the crack. (Before the cracking takes place, the entire shear resistance is provided by the masonry.)

The resistance provided by shear reinforcement is taken as the sum of tension forces developed in steel reinforcement (area A_v) which crosses the crack, as shown in Figure 2-15b. The number of reinforcing bars crossing the crack can be approximately taken equal to d_v/s.

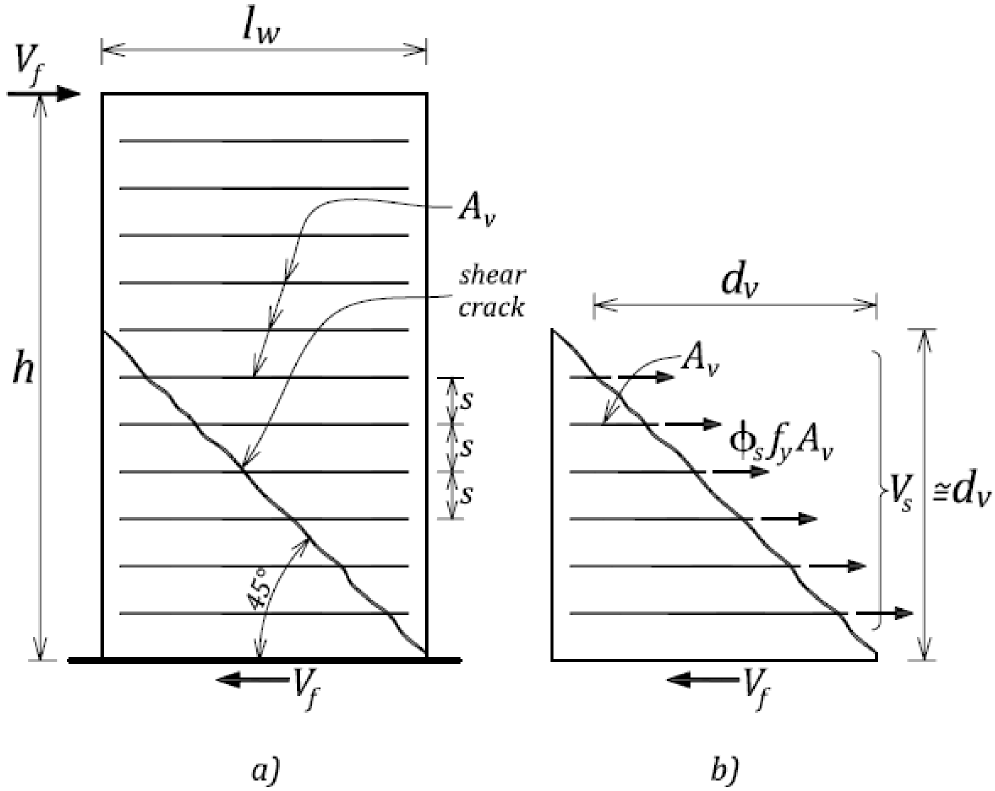


Figure 2-15. Steel shear resistance in flexural walls: a) wall elevation; b) free-body diagram showing reinforcement crossing a diagonal crack.

It appears that the steel reinforcement is less effective in resisting shear in masonry walls than in reinforced concrete walls. This may be due to a rather low masonry bond strength, so that not all bars crossing the assumed failure plane are fully stressed, plus the failure plane may be at an angle of less than 45° in this high moment region. Even in lightly reinforced masonry walls, horizontal reinforcement is less effective than in otherwise similar reinforced concrete walls. It is difficult to exactly estimate the contribution of steel reinforcement toward the shear resistance of masonry walls. Anderson and Priestley (1992) came to the conclusion that the contribution of steel shear reinforcement in a masonry wall is equal to 50% of the value expected in reinforced concrete walls. As a result, they proposed the following equation for the nominal steel shear resistance, V_s, (note that ϕ_s is equal to 1):

$$V_s = 0.5 A_v f_y \frac{d_v}{s}$$

CSA S304.1-04 uses the same V_s equation (4), except that the coefficient 0.6 is used instead of 0.5. Note also that, when 0.6 is multiplied by the ϕ_s value of 0.85, the resulting value is equal to $0.6 \times 0.85 = 0.51 \approx 0.5$.

The contribution of vertical reinforcement to shear resistance in masonry walls is not considered to be significant and it is not accounted for by the CSA S304.1-04 shear design equation. The analysis of experimental test data by Anderson and Priestley (1992) showed an absence of correlation between the wall shear resistance and the amount of vertical steel reinforcement.

2.3.2.2 Squat shear walls

10.10.1.3

Squat shear walls are characterized by a low height/length aspect ratio, h_w/l_w, less than unity. The factored shear resistance of squat shear walls, V_r, should be determined from the same equation as prescribed for flexural walls. To recognize the fact that shear resistance of masonry walls increases with a decrease in the height/length aspect ratio, CSA S304.1-04 prescribes an increased upper limit for the factored shear resistance as follows:

$$V_r \le \max V_r = 0.4\phi_m \sqrt{f'_m} b_w d_v \gamma_g (2 - \frac{h_w}{l_w}) \qquad \frac{h_w}{l_w} \le 1.0 \qquad (9)$$

Cl.10.10.1.3 also prescribes that this maximum shear resistance can be used only when it is ensured that the shear input to the wall is distributed along the entire length and that a failure of a portion of the wall is prevented. Application is subsequently discussed under Commentary.

Commentary

The first term in equation (9) is equal to the maximum V_r limit for flexural shear walls (equation 8). Equations (8) and (9) have the same value for a wall with the aspect ratio $h_w/l_w = 1.0$. The term $(2 - h_w/l_w)$ that accounts for the effect of wall aspect ratio has the minimum value of 1.0 for the aspect ratio of 1.0 and its value increases for squat walls – it is equal to 1.5 for the aspect ratio of 0.5.

Cl.10.10.1.3 prescribes that an increased maximum V_r limit for squat shear walls applies only when the designer can ensure that the shear input to the wall can be distributed along the entire wall length. Earthquake-induced lateral load in a masonry building is transferred from the floor or roof diaphragm into shear walls. Floor and roof diaphragms in masonry buildings range from flexible timber diaphragms to rigid reinforced concrete slab systems. The type of load transfer at the wall-to-diaphragm connection depends on the diaphragm rigidity (see Section 1.5.9.4 for more details).

CSA S304.1-04 Cl.10.15.1.3 requires that a bond beam be placed at the top of the wall, where the wall is connected to roof and floor assemblies. The bond beam therefore acts as a "transfer beam" that ensures a uniform shear transfer atop the wall, as shown in Figure 2-16a (this can be effectively achieved when the vertical reinforcement extends into the beam).

Shear forces are transferred from the top to the base of the wall by means of a compression strut. It should be noted that a majority of experimental studies used specimens with a rigid transfer beam cast on top of the wall, as discussed by Anderson and Priestley (1992). Provision of the top transfer beam (or an alternative means to apply shear force uniformly along the wall length) is required for seismic design of moderately ductile squat shear walls (Cl.10.16.6.2).

Where there is no transfer beam or bond beam at the top of the wall as shown in Figure 2-16b, a partial shear failure of the wall is anticipated. In such cases, the designer cannot take advantage of the increased maximum V_r limit for squat shear walls; the limit pertaining to flexural shear walls should be used instead.

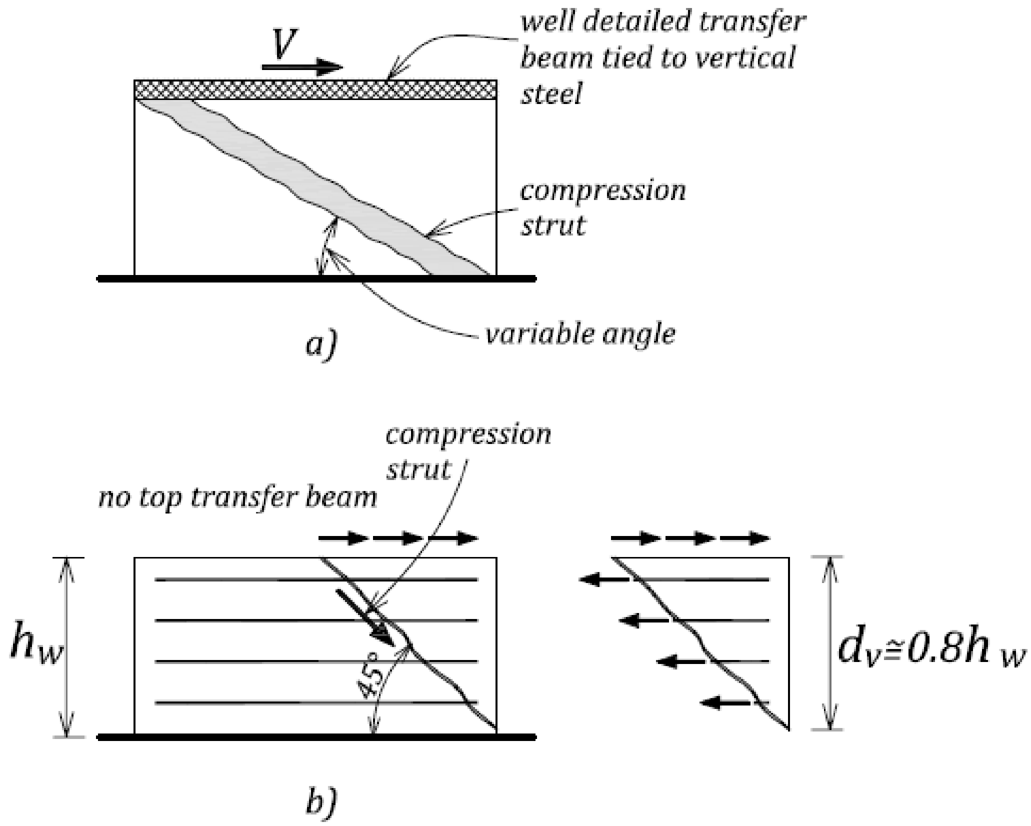


Figure 2-16. Shear failure mechanisms in squat shear walls: a) wall with the top transfer beam – a desirable failure mechanism; b) partial failure of a squat wall without the top beam.

2.3.3 Sliding Shear Resistance

Sliding shear failure may occur before walls fail in the flexural mode. Experimental studies (Shing et al., 1990) have shown that for squat walls sliding shear mechanism can control the failure and prevent the development of their full flexural capacity. This section discusses the sliding shear resistance provisions of CSA S304.1-04 for non-seismic conditions; seismic requirements related to sliding shear resistance will be discussed in Section 2.5.4.6.

10.10.4

Sliding shear failure can occur in both squat and flexural walls; however, it is much more common in squat walls having high shear resistance, V_r. Sliding shear resistance is usually checked at the foundation-to-wall interface (construction joint), but may need to be checked at other sections as well (especially upper portions of multi-storey flexural walls).

10.10.4.1

Sliding shear resistance is generally taken as a frictional coefficient times the maximum compressive force at the sliding plane. In accordance with CSA S304.1-04, the factored in-plane sliding shear resistance, V_r, shall be taken as:

$$V_r = \phi_m \mu P_2 \qquad (10)$$

where

μ is the coefficient of friction

= 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

= 0.7 for a masonry-to-smooth concrete or bare steel sliding plane

= other (where flashings reduce friction that resists sliding shear, a reduced coefficient of friction accounting for the flashing material properties should be used)

P_2 is the compressive force in the masonry acting normal to the sliding plane, normally taken as $P_2 = P_d + T_y$

$T_y = \phi_s A_s f_y$ the factored tensile force at yield of the vertical reinforcement of area A_s (yield stress f_y)

P_d = axial compressive load on the section under consideration, based on 0.9 times dead load, P_{DL}, plus any axial load acting from bending in coupling beams

Note that A_s denotes the total area of vertical reinforcement crossing the sliding plane for seismic design of limited ductility shear walls and moderately ductile squat shear walls. However, A_s denotes the area of reinforcement in the tension zone only for moderately ductile shear walls (Cl.10.16.5.3.2). Note that, when sliding takes place at the base of the wall, the vertical reinforcement is in the form of dowels. For more details refer to Section 2.5.4.6.

Commentary

When sliding begins, the sand grains in the mortar tend to ride up and over neighbouring particles causing the mortar to expand in the vertical direction. This creates tension (and ultimately yielding) in the vertical reinforcing bars at the interface (note that adequate anchorage of reinforcement on both sides of the sliding plane is necessary to develop the yield stress). As a result, a clamping force is formed between the support and the wall, normally taken equal to $\phi_s A_s f_y$, as shown in Figure 2-17. The shear is then transferred through friction at the interface along the compression zone of the wall.

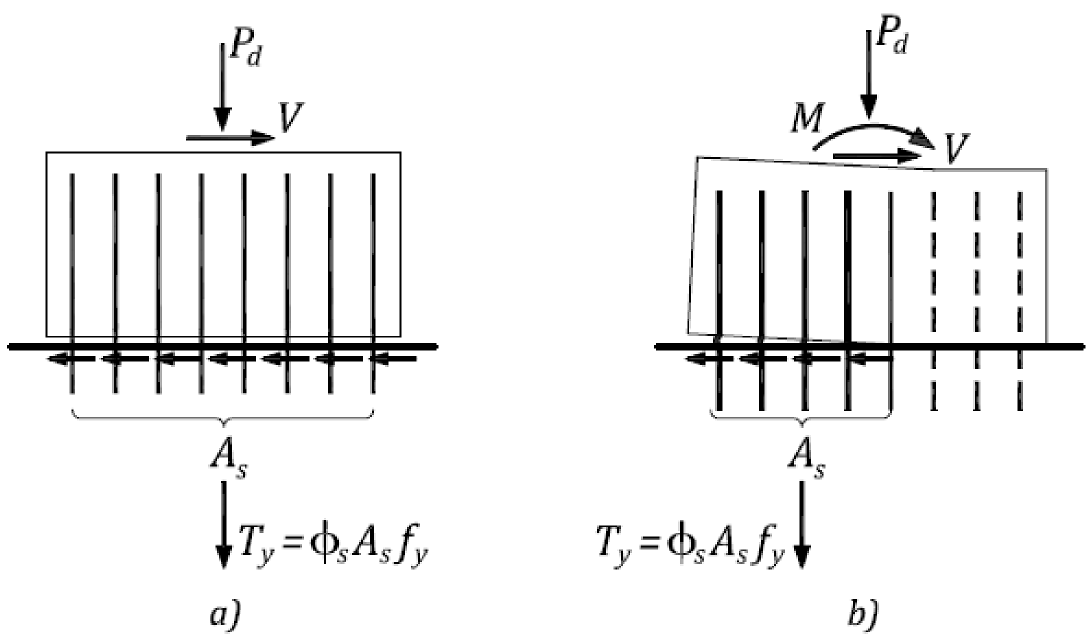


Figure 2-17. In-plane sliding shear resistance in masonry shear walls: a) conventional construction and limited ductility walls; b) moderately ductile walls.

In accordance with CSA S304.1-04, the maximum compression force, P_2, is usually considered to be equal to the axial load plus the yield strength of the reinforcement/dowels crossing the sliding plane. Since the reinforcement yields in tension, shear resistance of the dowels cannot be included. This assumption is appropriate for walls that are not expected to demonstrate significant ductility.

However, if a wall is subjected to ultimate moment capacity which causes yielding of the compression reinforcement, there is a tendency for this reinforcement to remain in compression to maintain the moment resistance, especially after the wall has been cycled into the yield range once or twice. Thus, when the compression steel remains in compression, and the normal force resisting sliding will be limited to the resultant force in the tension steel, T_y, as shown in Figure 2-17b. This assumption is included in seismic design requirements for moderately ductile walls (to be discussed in Section 2.5.4.6).

Presence of flashing at the base of the wall usually reduces the sliding shear resistance when the frictional coefficient for the flashing-to-wall interface is low (Anderson and Priestley, 1992).

2.3.4 In-Plane Flexural Resistance Due to Combined Axial Load and Bending

Seismic shear forces acting at floor and roof levels cause overturning bending moments in a shear wall, which reach the maximum at the base level. The theory behind the design of masonry wall sections subjected to effects of flexure and axial load is well established and the design methodology is essentially the same as that related to reinforced concrete walls.
Note that CSA S304.1-04 Cl.10.2.8 prescribes the use of reduced effective depth, d, for flexural design of *squat shear walls*, that is:

$$d = 0.67 l_w \leq 0.7h$$

This provision was introduced for the first time in the 2004 edition of CSA S304.1, in order to account for the deep beam-like flexural response of squat shear walls. This provision can be rationalized for non-seismic design, but it should not be used in seismic conditions as all the tension steel is expected to yield, as shown in Figure 2-17b. The wall design according to this provision could give the flexural capacity of the wall larger than permitted according to the capacity design approach.

For a detailed flexural design procedure the reader is referred to Appendix C (Section C.1.1).

2.4 Reinforced Masonry Walls Under Out-of-Plane Seismic Loading

2.4.1 Background

Seismic shaking in a direction normal to the wall causes out-of-plane wall forces that result in bending and shear stresses and may, ultimately, cause out-of-plane collapse of the walls. Note that out-of-plane seismic response of masonry walls is more pronounced at higher floor levels (due to larger accelerations) than in the lower portions of the buildings, as shown in Figure 2-18. When walls are inadequately connected to top and bottom supports provided by floor and/or roof diaphragms, out-of-plane failure is very likely, and may also lead to a diaphragm failure. For more details on wall-to-diaphragm connections, the reader is referred to Section 2.6.7. The design of masonry walls for shear and flexure due to the effects of out-of plane seismic loads is discussed in this section.

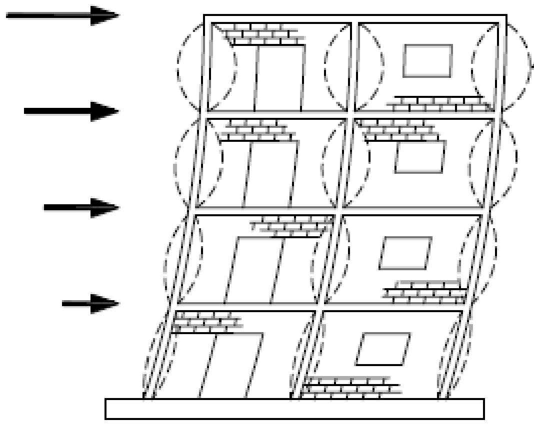


Figure 2-18. Out-of-plane vibration of walls (Tomazevic, 1999, reproduced by permission of the Imperial College Press).

2.4.2 Out-of-Plane Shear Resistance

10.10.2

The factored out-of-plane shear resistance, V_r, shall be taken as:

$$V_r = \phi_m (v_m \cdot b \cdot d + 0.25 P_d) \qquad (11)$$

where

$v_m = 0.16\sqrt{f'_m}$ *MPa units* (Cl.10.10.1.4)

with the following upper limit,

$$V_r \leq \max V_r = 0.4\phi_m \sqrt{f'_m}(b \cdot d) \qquad (12)$$

where
d is the distance from extreme compression fibre to the centroid of tension reinforcement, and
b is the cumulative width of cells and webs within a length not greater than four times the actual wall thickness $(4 \times t)$ around each vertical bar (for running bond), as shown in Figure 2-19a. Note that, for the purpose of this provision, the webs are the cross-walls connecting the face shells of a hollow or semi-solid concrete masonry unit or a hollow clay block (S304.1 Cl.10.10.2).

Commentary

Note that the equation for masonry shear resistance, V_m, is the same for shear walls subjected to in-plane and out-of-plane seismic loading. There is no V_s contribution because the horizontal reinforcement is provided only in the longitudinal direction and it does not contribute to the out-of-plane shear resistance.

In partially grouted walls, the out-of-plane shear design should be performed using a T-shaped wall section, where b denotes the web width (see Figure 2-19a).

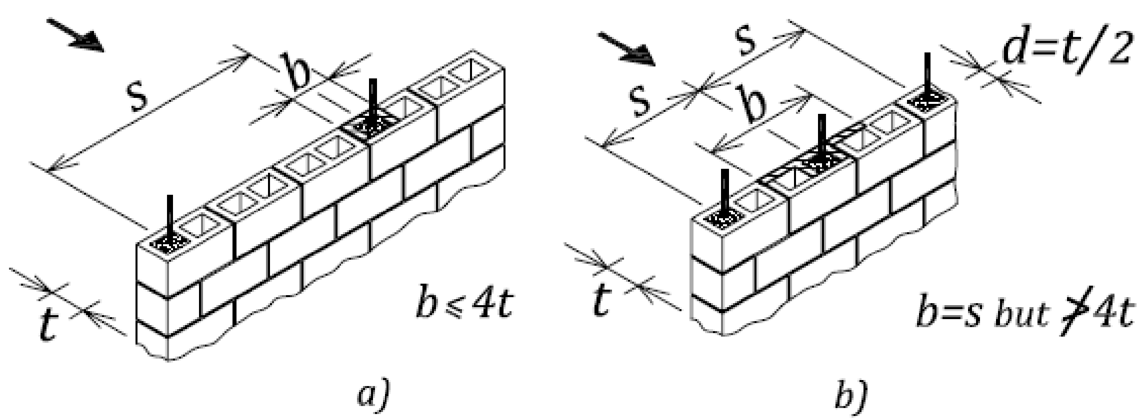


Figure 2-19. Effective width, b, for out-of-plane seismic effects: a) shear, and b) flexure.

2.4.3 Out-of-Plane Sliding Shear Resistance

10.10.4.2

The factored out-of-plane sliding shear resistance, V_r, is calculated from the following equation using the shear friction concept:

$V_r = \phi_m \mu C$ *(13)*

where
μ = the coefficient of friction (same as for the in-plane sliding shear resistance)
C = compressive force in the masonry acting normal to the sliding plane, taken as
$C = P_d + T_y$
T_y = the factored tensile force at yield of the vertical reinforcement detailed to develop yield strength on both sides of the sliding plane. In determining the out-of-plane sliding shear resistance, the entire vertical reinforcement should be taken into account in determining the factored tensile yield force, T_y, irrespective of the wall class and the associated ductility level.

For more details refer to the discussion on the sliding shear resistance of shear walls under in-plane seismic loading (Section 2.3.3).

2.4.4 Out-of-Plane Section Resistance Due to Combined Axial Load and Bending

Masonry walls subjected to out-of-plane seismic loading need to be designed for combined effects of bending and axial gravity loads. For flexural design purposes, wall strips of predefined width b (S304.1-04 Cl.10.6.1) are treated as beams spanning between the lateral supports. When the walls span in the vertical direction, floor and/or roof diaphragms provide lateral supports. Walls can also span horizontally, in which case lateral supports need be provided by cross walls or pilasters. For detailed design procedures, the reader is referred to Section C.1.2 in Appendix C. It should be noted that, for the purpose of out-of-plane seismic design, the permitted maximum compressive strain in the masonry is equal to 0.003 (note that this is an arbitrary value set for the purpose of the analysis). CSA S304.1 does not require ductility check, because the mechanism of failure is different for the in-plane and out-of-plane seismic resistance and the wall is not expected to undergo significant rotations at the locations of maximum bending moments. Very large curvatures would be required to cause compression failure of the masonry, corresponding to a high strain gradient over a very small length (equal to the wall thickness). Consequently, there is no need to use the reduced compressive strain limit of 0.0025 for this load condition.

10.6.1

For the case of out-of-plane bending, the effective compression zone width, b, used with each vertical bar in the design of walls with vertical reinforcement shall be taken as the lesser of (see Figure 2-19 b)

a) spacing between vertical bars s, or
b) four times the actual wall thickness $(4\times t)$

Note that the discussion on out-of-plane stability issues is outside the scope of this document and it is covered elsewhere (see Drysdale and Hamid, 2006).

2.5 Seismic Design Considerations for Reinforced Masonry Shear Walls

2.5.1 Background

The focus of this section is mainly on the seismic design and detailing requirements for different classes of ductile reinforced masonry shear walls. General seismic design requirements for ductile shear walls are stated in Cl. 10.16. In the 1994 edition of this standard (CSA S304.1-94), seismic design requirements for ductile reinforced shear walls were included in Cl. 5.2.2, 6.3.3, and Appendix A. Changes in seismic design provisions between the two editions of CSA S304.1 will also be discussed in this section. Shear walls with conventional construction do not require seismic detailing since these walls are not designed for ductile performance.

It should be noted that NBCC 2005 also identifies moment-resisting frames with conventional construction as a possible masonry SFRS, however seismic design of moment-resisting masonry frames is beyond the scope of this document.

2.5.2 Capacity Design Approach

10.16.3.3

According to the design approach stated in Cl.10.16.3.3, a ductile reinforced masonry shear wall must be designed to resist a shear force not less than the shear that is present when the wall develops a plastic hinge mechanism.

Every structure or a structural component has several possible modes of failure, some of which are ductile while others are brittle. Satisfactory seismic response of structures requires that brittle failure modes be avoided. This is accomplished through the application of a *capacity design approach*, which has been used for seismic design of reinforced concrete structures since the 1970's (Park and Paulay, 1975). The objective of the capacity design approach is to force the structure to yield in a ductile manner without failing at the expected displacements (including other components of the structure, such as columns). At the same time, the rest of the structure needs to remain strong enough, say in shear, or flexible enough not to fail under gravity loads at these displacements.

This concept can be explained using an example of a chain shown in Figure 2-20, which is composed of brittle and ductile links. When subjected to force, F, and if the brittle link is weakest, the chain will fail suddenly without significant deformation (see Figure 2-20a). If a ductile link is the weakest, the chain will show significant deformation before failure and may not fail or break if the deformation is not too great (see Figure 2-20b). The structural designer is responsible for ensuring that the structure experiences a desirable ductile response when exposed to the design earthquake, that is, an earthquake of expected intensity for the specific building site location.

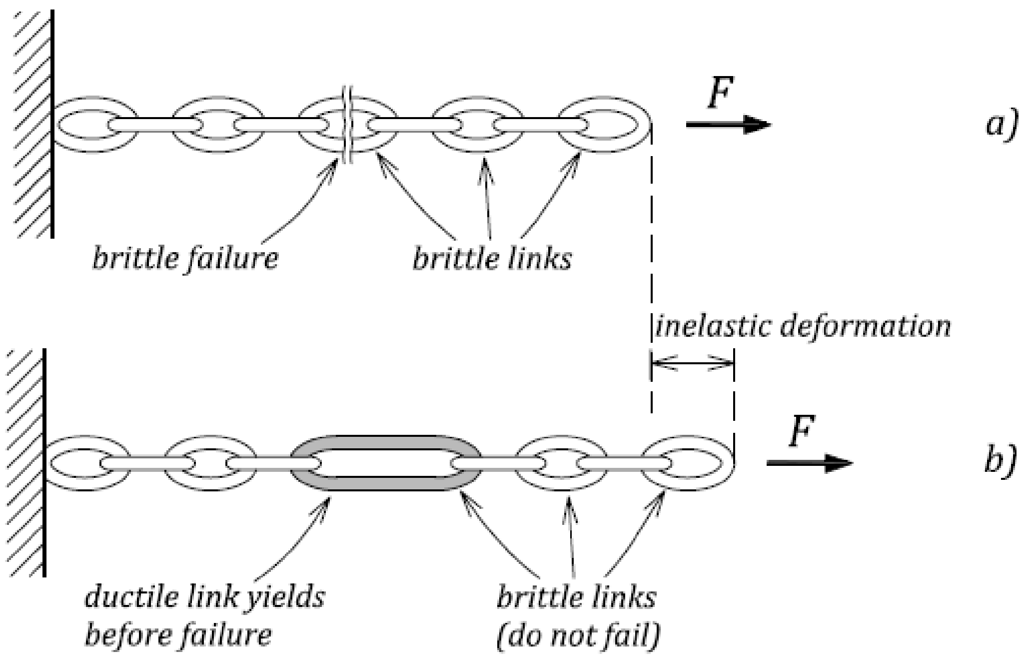


Figure 2-20. Chain analogy for capacity design: a) brittle failure; b) ductile failure.

The capacity design approach can be applied to seismic design of reinforced masonry shear walls. The key failure modes in reinforced masonry walls include flexural failure (which is ductile and therefore desirable in seismic conditions) and shear failure (which is brittle and should be avoided in most cases). For a detailed discussion of masonry failure modes refer to Section 2.3.1.

Note that the following three resistance "levels" are used in seismic design of masonry shear walls:

- *Factored resistances* M_r and V_r, determined using appropriate material resistance factors, that is, ϕ_m = 0.6 and ϕ_s = 0.85, and specified material strength;
- *Nominal resistances* M_n and V_n, determined without using material resistance factors, that is, ϕ_m = 1.0 and ϕ_s = 1.0, and specified material strength;
- *Probable resistances* M_p and V_p, determined without using material resistance factors; stress in the tension reinforcing is taken equal to $1.25 f_y$, and the masonry compressive strength is equal to f'_m.

In relation to the probable resistance parameters discussed above, it needs to be clarified that the flexural resistance of a masonry shear wall is usually governed by the yield strength of the reinforcement, f_y, while the masonry compressive strength, f'_m, has a much smaller influence.

Thus the probable resistances are determined by taking the masonry strength equal to f_m' and the real yield strength of the reinforcement equal to 1.25 the specified strength, that is, $1.25 f_y$.

Consider a masonry shear wall subjected to an increasing lateral seismic force, V, and the corresponding deflection shown in Figure 2-21a. The wall has been designed for a “design shear force” shown by a horizontal line. However, the actual wall capacity typically exceeds the design force, and the wall is expected to deform either in a flexural or shear mode at higher load levels. Conceptual force-deflection curves corresponding to shear and flexural failure mechanisms are also shown on the figure. These curves are significantly different: a shear failure mechanism is characterized by brittle failure at a small deflection, while a ductile flexural mechanism is characterized by significant deflections before failure takes place.

The earthquake will cause significant lateral deflections, which are more or less independent of the strength. If the governing failure mode corresponding to the lowest capacity occurs at a smaller deflection, the wall will fail in that mode. For example, the wall shown in Figure 2-21a is expected to experience shear failure since the maximum force corresponding to shear failure is lower than the force corresponding to flexural failure.

Consider a wall which is designed to fail in shear when the shear resistance, V_A, and corresponding displacement Δ_A have been reached, and to fail in flexure when the shear force, V_B, and corresponding displacement Δ_B have been reached (see Figure 2-21b). If the wall is weaker in flexure than in shear, that is, $V_B < V_A$, the shear failure will never take place. In this case, a ductile link corresponding to the flexural failure is the weakest and governs the failure mode. Such a wall will experience significant deflections before the failure (note that $\Delta_B \geq \Delta_A$); this is a desirable seismic performance.

However, suppose that the wall flexural resistance is higher (this is also known as “flexural overstrength”) and now corresponds to moments associated with the shear force, V_C, as shown in Figure 2-21c. Now the wall will fail in shear at the force, V_A, and will never reach the force V_C. This is not a desirable wall design, since shear failure is brittle and sudden and should be avoided. Thus, it is important that the member shear strength be greater than its flexural overstrength, as we will discuss later in this section.

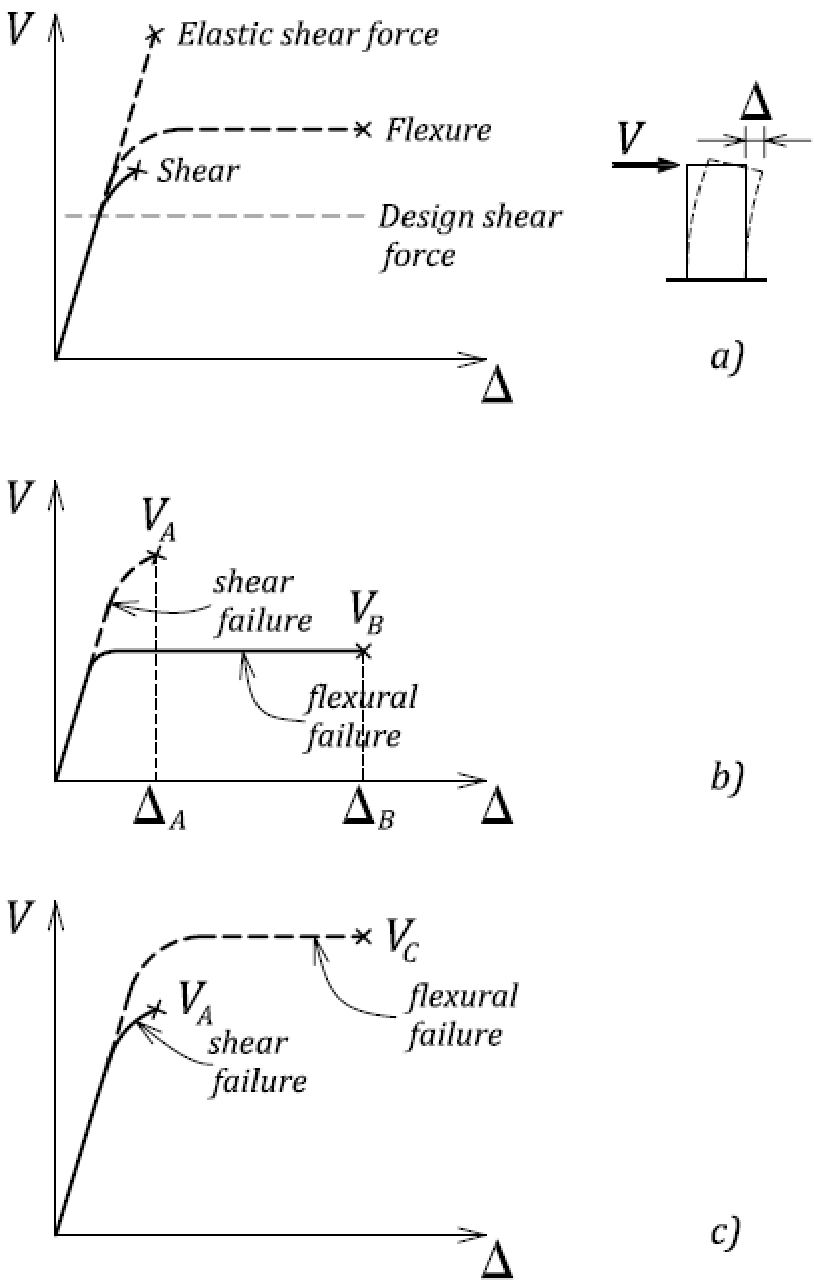


Figure 2-21. Shear force-deflection curves for flexural and shear failure mechanisms: a) a possible design scenario; b) flexural mechanism governs; c) shear mechanism governs (adapted from Nathan).

The last example demonstrates that making the wall "stronger" can have unintended adverse effects, and can change the failure mode from ductile flexural mode (good) to brittle shear mode (bad). Thus a designer should not indiscriminately increase member moment capacity without also increasing its shear capacity. According to the capacity design approach, ductile flexural failure will be assured when the shear force corresponding to the upper bound of moment resistance at the critical wall section is less than the shear force corresponding to the lower bound shear resistance of the shear failure mechanism. This will be explained with an example of the shear wall shown in Figure 2-22.

When the moment at the base is equal to the nominal moment resistance, M_n (this is considered to be an upper bound for the moment resistance value and it is explained below), the corresponding shear force acting at the effective height is equal to

$$V_{nb} = M_n / h_e$$

or

$$V_{nb} = M_n * (V_f / M_f)$$

as shown in Figure 2-22a. V_{nb} denotes the resultant shear force corresponding to the development of nominal moment resistance, M_n, at the base of the wall. To ensure the development of a ductile flexural failure mode, V_{nb} must be less than the corresponding factored shear resistance, V_r, as shown in Figure 2-22b.

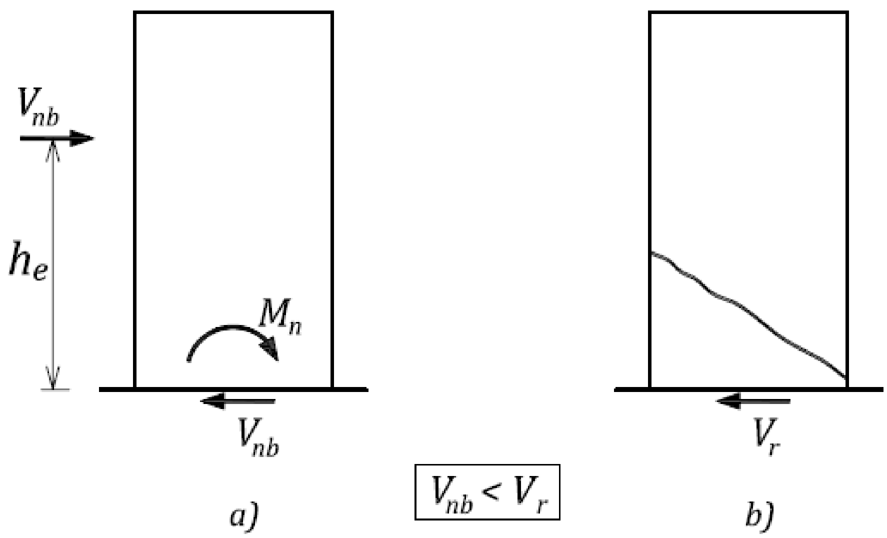


Figure 2-22. Comparison of shear forces at the base of the wall: a) shear force corresponding to the nominal flexural resistance, and b) shear force equal to the shear resistance.

The key capacity design concepts as applied to the design of masonry structures can be summarized as follows:

1. For the design of reinforced masonry walls, the factored shear resistance, V_r, should be greater than the shear due to effects of factored loads, but not less than the shear corresponding to the development of nominal moment capacity, M_n, of the wall system at its plastic hinge location (see Section 2.5.4.2 for detailed discussion on plastic hinges). The nominal moment capacity need not be taken larger than the load calculated with design load combinations that include earthquake effects calculated using $R_d R_o$ equal to 1.0.

2. It is also important that other structural members which are not a part of the SFRS are able to undergo the same lateral displacements as the SFRS members without experiencing brittle failure.

Although CSA S304.1-04 Cl. 10.16.3.3 requires that the capacity design approach should be applied to ductile masonry walls, additionally, it is recommended that this approach be applied to all reinforced masonry shear walls. As a minimum, factored shear resistance, V_r, should be not less than the shear corresponding to the factored moment resistance, M_r, of the wall system at its plastic hinge location.

2.5.3 Ductile Seismic Response

A prime consideration in the seismic design is the need to have a structure capable of deforming in a ductile manner when subjected to several cycles of lateral loading well into the inelastic range. *Ductility* is a measure of the capacity of a structure or a member to undergo deformations beyond yield level while maintaining most of its load-carrying capacity. Ductile structural members are able to absorb and dissipate earthquake energy by inelastic (plastic) deformations, which are usually associated with permanent structural damage.

The concept of ductility and ductile response was introduced in Section 1.4.3. Key terms related to ductile seismic response of masonry shear walls, including ductility ratio, curvature, plastic hinge, etc. are discussed in detail in Section B.2. It is very important for a structural designer to have a good understanding of these concepts before proceeding with the seismic design and detailing of ductile masonry walls in accordance with CSA S304.1-04.

2.5.4 CSA S304.1-04 Seismic Design Requirements

2.5.4.1 Classes of reinforced masonry shear walls

Table 4.1.8.9 of NBCC 2005 identifies the following five classes of masonry walls based on the expected seismic performance quantified by means of the ductility-related force modification factor, R_d (see also Section 1.5.5):

1. Unreinforced masonry and other masonry structural systems not listed below ($R_d = 1.0$)
2. Shear walls with conventional construction ($R_d = 1.5$)
3. Limited ductility shear walls ($R_d = 1.5$)
4. Moderately ductile shear walls ($R_d = 2.0$)
5. Moderately ductile squat shear walls ($R_d = 2.0$) (note that this wall class was not identified in NBCC 2005 Table 4.1.8.9, however specific design and detailing provisions are stipulated by S304.1-04 Cl.10.16.6).

The last three classes are referred to as “ductile shear walls”. Although shear walls with conventional construction and shear walls of limited ductility have the same $R_d = 1.5$ value, the difference between these walls is that the limited ductility walls have additional detailing requirements, and so can be used in taller structures. The same value of overstrength factor, R_o, of 1.5 is prescribed for all the above listed wall classes except for unreinforced masonry where R_o is equal to 1.0.

CSA S304.1-04 Clause 4.6 outlines the classes of reinforced masonry walls, while the seismic design provisions are prescribed in Clause 10.

New seismic design provisions for reinforced masonry walls in NBCC 2005 and CSA S304.1-04 are summarized below (note that new terms were introduced to provide consistency with the concrete design standard, CSA A23.3-04):

1. New term “shear walls with conventional construction” (previously “reinforced masonry”)
2. New term “limited ductility shear walls” (S304.1-04, Cl.10.16.4)
3. New term “moderately ductile shear walls” (S304.1-04, Cl.10.16.5); note that S304.1-94 used term “masonry with nominal ductility”
4. New requirements for “moderately ductile squat shear walls” (S304.1-04, Cl. 10.16.6)
5. Height-to-thickness ratio restrictions introduced for ductile shear walls: limited ductility shear walls, moderately ductile shear walls, and moderately ductile squat shear walls.

Seismic design and detailing requirements for various masonry Seismic Force Resisting Systems (SFRSs) are summarized in Table 2-1. In accordance with NBCC 2005 Sent.4.1.8.1.1), seismic design must be performed when $S(0.2) > 0.12$. Also, it is permissible to use unreinforced masonry constructions at sites where $I_E F_a S_a(0.2) < 0.35$ (S304.1-04, Cl.4.5.1). Minimum CSA S304.1-04 seismic reinforcement requirements for masonry walls are summarized in Table 2-2.

Note that squat shear walls are most common in low-rise masonry construction, ranging from warehouses, school buildings, and fire halls. Some of these buildings, for example fire halls, are considered to be post-disaster facilities according to NBCC 2005. A new restriction has been introduced in NBCC 2005 (Sent. 4.1.8.10.2), by which post-disaster facilities require an SFRS with R_d of 2.0 or higher. An implication of this provision is that squat shear walls in post-disaster buildings be designed following the CSA S304.1-04 provisions for “moderately ductile squat shear walls”.

Table 2-1. Summary of Seismic Design and Detailing Requirements for Masonry SFRSs in CSA S304.1-04

Type of SFRS	**Common applications**	R_d	R_o	**Expected seismic performance**	**Summary of CSA S304.1-04 design requirements**	**CSA S304.1 reinforcing and detailing requirements**
Unreinforced masonry	Low-rise buildings located in low seismicity regions	1.0	1.0	Potential to form brittle failure modes	▪ Can be used only at sites where $I_E F_a S_a(0.2) < 0.35$ ▪ Walls must have factored shear and flexural resistances greater than or equal to corresponding factored loads	Reinforcement not required
Shear walls with conventional construction	Used for most building applications	1.5	1.5	Design to avoid soft stories or brittle failure modes	Walls must have factored shear and flexural resistances greater than or equal to corresponding factored loads	Minimum seismic reinf. requirements (Cl.10.15.2.2) apply if $I_E F_a S_a(0.2) \geq 0.35$ otherwise follow minimum non-seismic reinf. requirements (Cl.10.15.1.1)
Limited ductility shear walls	Used only when required to comply with the NBCC 2005 height restrictions (Table 4.1.8.9)	1.5	1.5	Limited dissipation of earthquake energy by flexural yielding in specified locations; shear failure to be avoided	▪ Can be used for shear wall design when $h_w / l_w \geq 1.0$ ▪ Walls to be designed using factored moment resistance such that plastic hinges develop without shear failure and local buckling ▪ Sliding shear failure at joints to be avoided ▪ Expected ductility level to be verified ▪ Wall height-to-thickness ratio restrictions prescribed	Minimum seismic reinforcement requirements (Cl.10.15.2.2) must be satisfied, as well as seismic detailing requirements for limited ductility walls (Cl.10.16.4)

Type of SFRS	**Common applications**	R_d	R_o	**Expected seismic Performance**	**Summary of CSA S304.1-04 design requirements**	**CSA S304.1 reinforcing and detailing requirements**
Moderately ductile shear walls	Used for post-disaster or high risk buildings or where $R_d = 2.0$ is desired	2.0	1.5	Dissipation of earthquake energy by ductile flexural yielding in specified locations; shear failure to be avoided	▪ Walls to be designed using factored moment resistance such that plastic hinges develop without shear failure and local buckling 50% reduction in masonry resistance for V_r calculations ▪ Sliding shear failure at joints to be avoided (additional requirements compared to limited ductility walls) ▪ Expected ductility level to be verified ▪ Wall height-to-thickness ratio restrictions more stringent than limited ductility walls	Minimum seismic reinforcement requirements (Cl.10.15.2.2) must be satisfied, as well as seismic detailing requirements for moderately ductile walls (Cl.10.16.5)
Moderately ductile <u>squat</u> shear walls	Used for post-disaster buildings or where $R_d = 2.0$ is desired	2.0	1.5	Top transfer beam to ensure uniform shear transfer along the wall length; some flexural yielding expected	▪ Walls to be designed using factored moment resistance; shear failure and local buckling to be avoided ▪ Sliding shear failure at joints to be avoided ▪ Wall height-to-thickness ratio restrictions less stringent than limited ductility walls	Minimum seismic reinforcement requirements (Cl.10.15.2.2) must be satisfied, as well as special reinforcement requirements for moderately ductile squat shear walls

2.5.4.2 Plastic hinge region

10.16.4.1.1 10.16.5.2.1

The required extent of the plastic hinge region above the base of a shear wall in the vertical direction (also referred to as plastic hinge length, l_p) prescribed by CSA S304.1-04 is as follows (see Figure 2-23):

1. Limited ductility shear walls (Cl.10.16.4.1.1):
   l_p = greater of $l_w / 2$ or $h_w / 6$
2. Moderately ductile shear walls (Cl.10.16.5.2.1):
   l_p = greater of l_w or $h_w / 6$

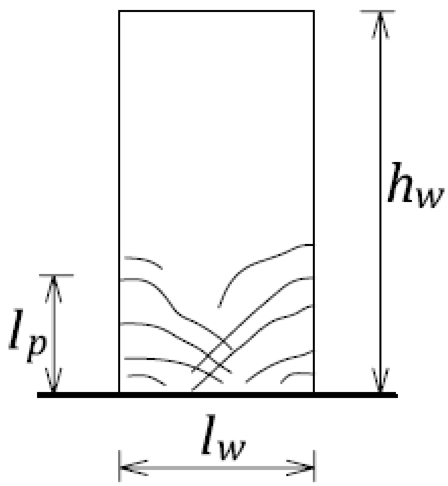


Figure 2-23. Plastic hinge length l_p.

10.16.4.1.3

Masonry within the plastic hinge region must be fully grouted. This provision applies to limited ductility and moderately ductile shear walls in running bond. Note that this provision applies to moderately ductile shear walls by way of Cl.10.16.5.1. Refer to Section 2.6.3.5 for the discussion on plastic hinge requirements related to stack pattern walls.

Commentary

According to CSA S304.1-04 Cl. 10.16.2, the *plastic hinge* is the region of the member where inelastic flexural curvatures occur. In reinforced masonry shear walls which are continuous along the building height, this region is located at the base of the walls, as shown in Figure 2-23. Plastic hinge length can be defined as a fraction of the wall height. In taller flexural walls (three stories or higher), this region can be up to one storey high (usually located at the first storey level). In low-rise buildings, this length is smaller, but it does exist even in squat shear walls when they are subjected to combined effects of axial load and bending and show flexure-dominated response.

The ability of a plastic hinge to sustain these plastic deformations will determine whether a structural member is capable of performing at a certain ductility level. The extent of the plastic hinge region is usually termed the *plastic hinge length,* l_p , although in the case of vertical elements such as walls and columns, it would be more appropriate to refer to it as the plastic hinge height. The l_p value depends on the moment gradient, wall height, wall length, and the level of axial load.

Note that the CSA S304.1-prescribed plastic hinge length values are intended for detailing purposes, and that smaller l_p values should be used for curvature and deflection calculations. There are a few different equations for estimating the value of l_p to be used in curvature calculations, and the recommended on is that proposed by Corley (1966):

$$l_p = 0.5d + 0.032\,h_w \big/ \sqrt{d}$$

where $d = 0.8l_w$ for rectangular walls

CSA S304.1-04 provisions for plastic hinge length, to be used in detailing, are in line with the research findings and codes in other countries. For example, in the New Zealand Masonry Standard NZS 4230:2004 (SANZ, 2004), Cl. 7.4.3 prescribes the plastic hinge length to be the greater of l_w, $h_w/6$, or 600 mm. Findings of a research study by Shing et al.(1990a) indicated that the plastic hinge length is in the order of $h_w/6$.

CSA S304.1-04 plastic hinge length requirements for moderately ductile shear walls (Clause 10.16.5.2.1) are the same as the corresponding CSA S304.1-94 requirements for shear walls with nominal ductility (Clause A5.1).

Design and detailing of plastic hinge regions in ductile masonry shear walls is critical and will be discussed in the following sections. These regions are usually heavily reinforced, and it is critical to ensure proper anchorage of reinforcement. Open-end H-blocks may simplify construction in these regions.

Plastic hinge regions of ductile masonry walls must be fully grouted. Observations from past damaging earthquakes (e.g. 1994 Northridge, California earthquake, Magnitude 6.7) that caused damage to masonry buildings, have shown that the quality of grout placement, and the bond of the grout to the masonry units and reinforcement have a strong influence on the performance of reinforced masonry structures. Some reinforced block walls with large voids around reinforcing bars suffered severe damage (TMS, 1994). CSA S304.1-04 grouting requirements for ductile masonry walls are the same as S304.1-94 requirements related to shear walls with nominal ductility (Clause A5.3). Grout in accordance with CSA A179-04, “Mortar and Grout for Unit Masonry”, offers sufficient strength.

2.5.4.3 Ductility check

10.16.4.1.4 10.16.5.2.3

CSA S304.1-04 prescribes the following two ductility requirements for reinforced masonry shear walls:

1. the neutral axis depth/wall length ratio, c/l_w , should be within the following limits:

   a) For limited ductility shear walls (Cl.10.16.4.1.4):

   $c/l_w < 0.2$ when $h_w/l_w < 6$

b) For moderately ductile shear walls (Cl.10.16.5.2.3):

$c/l_w < 0.2$ when $h_w/l_w < 4$

$c/l_w < 0.15$ when $4 \le h_w/l_w < 8$

2. if these requirements are not satisfied, the maximum compressive strain in the masonry in the plastic hinge region must be shown to not exceed 0.0025.

Commentary

For the purpose of the ductility check, the strain level in the masonry compression zone is limited to 0.0025. The intent is to limit deformations and the related damage in the highly stressed zone of a wall section.

Whether a structural member is capable of sustaining inelastic deformations consistent with an expected displacement ductility ratio, μ_Δ, will depend on the ability of its plastic hinge region to sustain corresponding plastic rotations. Plastic hinge rotations will depend on available curvature ductility, μ_φ, and the expected plastic hinge length. Refer to Section B.2 for a detailed explanation of curvature ductility and the relationship between curvature ductility and displacement ductility ratio.

It is important for a structural designer to have a sense for curvature ductility and its effect upon the ductile seismic performance. For example, a wall section shown Figure 2-24a is lightly reinforced and has a small axial compression (or tension) load. A small flexural compression zone will be required due to the light reinforcement, thus the neutral axis depth, c_1, will be small relative to the wall length (note the corresponding strain distribution - line 1 shown in Figure 2-24b). As a result, curvature, which is the slope of line 1, will be large and usually adequate to accommodate the plastic hinge rotations imposed on a structure during a major earthquake. However, when the wall is heavily reinforced and has significant axial compression load, a large flexural compression zone will be required, thus resulting in a relatively large neutral axis depth, c_2, as shown in Figure 2-24b (note the corresponding strain distribution - line 2 on the same diagram). For the same maximum strain in the concrete, the curvature (given by the slope of line 2) is much less than for lightly loaded wall. Thus the curvature ductility of the lightly loaded wall is much greater than the heavily loaded wall.

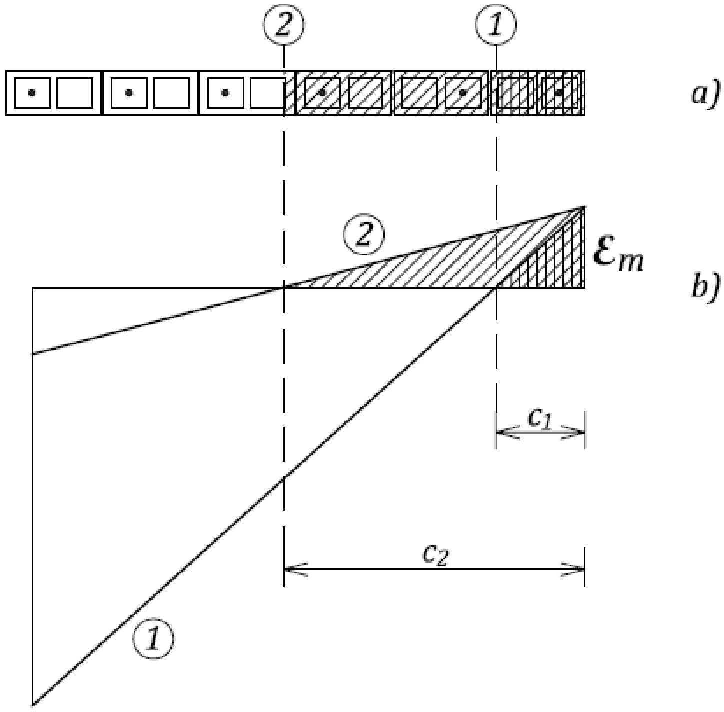


Figure 2-24. Strain distribution in a reinforced masonry wall at the ultimate: a) wall section; b) strain distribution.

Therefore, the ratio of neutral axis depth, c, relative to the wall length, l_w, that is, c/l_w ratio, is an indicator of the curvature ductility in a structural component. The c/l_w limits for ductile shear walls prescribed by CSA S304.1-04 cover most cases and save designers from performing time-consuming ductility calculations.

Where the c/l_w limit is not satisfied for a specific design, the designer may undertake detailed calculations to confirm that the ductility requirements have been met. Refer to Section B.2 for further guidance on detailed calculations related to the ductility requirements, and also Example 5a in Chapter 4 for a design application. In order to meet the S304.1-04 ductility requirements, the designer may want to consider an increase in the masonry strength or the wall thickness, or use flanged shear walls. Flanged wall sections will be discussed in Section 2.6.6.

CSA S304.1-94 ductility check

CSA S304.1-94 Clause A7 required a ductility check for shear walls with nominal ductility. The maximum masonry compression strain of 0.0025 was the same as the 2004 standard. The neutral axis depth requirement stated that

$$c/l_w < 0.2 \text{ when } h_w/l_w < 3$$

It can be concluded that no significant changes were made to the ductility check in CSA S304.1-04. The same c/l_w limit of 0.2 applies to both moderately ductile and limited ductility walls for h_w/l_w ratio values in the order of 4.0 or less, which is characteristic of low- to medium-rise masonry buildings.

As a reference, a discussion on the ductility check provisions of international standards is included in Section B.3.

2.5.4.4 Wall height-to-thickness ratio restrictions

10.16.4.1.2 10.16.5.2.2 10.16.6.3

CSA S304.1-04 prescribes the following height-to-thickness (h/t) limits for the compression zone in plastic hinge regions of ductile shear walls:

1. *Conventional construction*
   Slenderness limits and design procedures for masonry walls need to be followed (Cl.10.7.3.3) - it is possible to design walls with kh/t ratio greater than 30
2. *Limited ductility shear walls* (Cl.10.16.4.1.2):
   $h/(t+10) < 18$
3. *Moderately ductile shear walls* (Cl.10.16.5.2.2):
   $h/(t+10) < 14$
4. *Moderately ductile <u>squat</u> shear walls* (Cl.10.16.6.3):
   $h/(t+10) < 20$ (unless it can be shown for lightly loaded walls that a more slender wall is satisfactory for out-of-plane stability).

Note that h denotes the unsupported wall height (between the adjacent horizontal supports), kh denotes the effective buckling length, and t denotes the actual wall thickness (e.g. 140 mm, 190 mm, 240 mm, etc.).

Commentary

The purpose of this provision is to prevent instability due to out-of-plane buckling of shear walls when subjected to combined effects of in-plane axial loads and bending moments, as shown in Figure 2-25. This phenomenon is associated not only with compression in the masonry, but also with the compression stresses in the flexural reinforcement that has previously experienced large inelastic tensile strains. According to Paulay (1986), this instability occurs when the neutral axis depth, c, is large, as illustrated in Figure 2-24 (see depth c_2), and the plastic hinge region at the base of the wall (length l_p) is large (one storey high or more); this is characteristic of taller reinforced masonry shear walls.

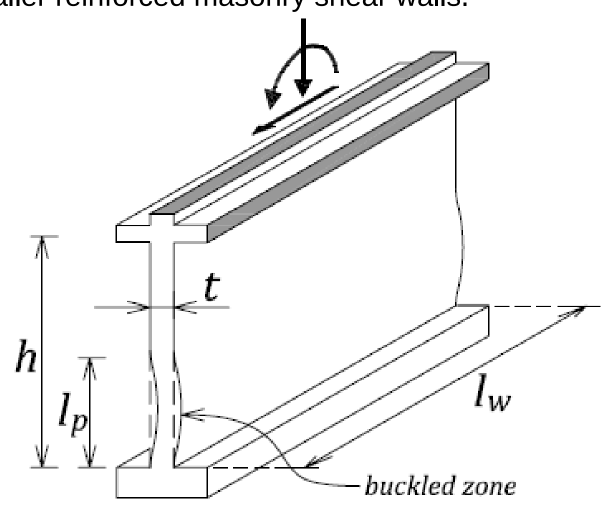


Figure 2-25. Out-of-plane instability in a shear wall subjected to in-plane loads (adapted from Paulay and Priestley, 1993, reproduced by permission of the American Concrete Institute).

A rational explanation for this phenomenon was first presented by Paulay (1986). When the wall experiences large curvature ductility, large tensile strains will be imposed on vertical bars placed at the extreme tension edge of the section. At this stage, uniformly spaced horizontal cracks of considerable width develop over the plastic hinge length (see Figure 2-26a). During the subsequent unloading, the tensile stresses in these bars reduce to zero. Change in the lateral load direction will eventually cause an increase in the compression stresses in these bars. Unless the cracks close, the entire internal compression within the section must be resisted by the vertical reinforcement, as shown in Figure 2-26b and d. At that stage, out-of-plane displacements may increase rapidly as the stiffness of the vertical steel to lateral deformation is small, thereby causing the out-of-plane instability. However, if the cracks close before the entire portion of the wall section previously subjected to tension becomes subjected to compression, masonry compressive stresses will develop in the section, the stiffness to lateral deformation is increased and the instability may be avoided (see Figure 2-26c and e). Refer to Section B.4 for a detailed discussion of the wall height-to-thickness ratio restrictions, and the analysis procedure developed by Paulay and Priestley (1992, 1993).

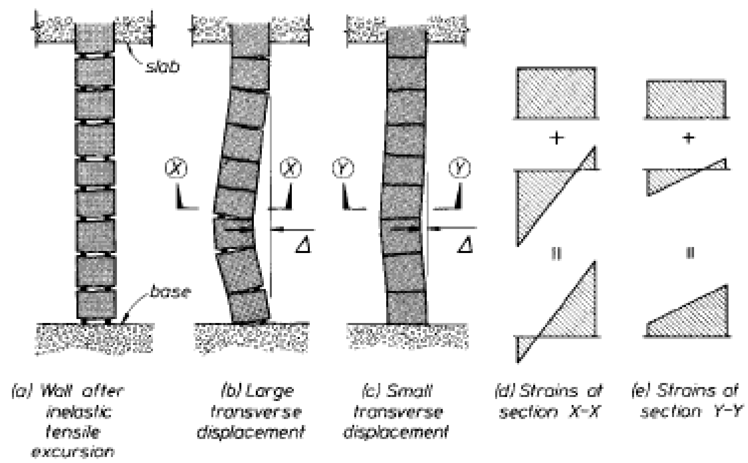


Figure 2-26. Deformations and strain patterns in a buckled zone of a wall section (Paulay,1986, reproduced by permission of the Earthquake Engineering Research Institute).

<u>Implications for seismic design</u>
This provision was first introduced in the 1994 edition of CSA S304.1 (Cl.A5.2 related to "nominally" ductile walls) and it is identical to the current CSA S304.1-04 provision for "moderately" ductile walls.

The height-to-thickness restrictions have significant effect on the required wall thickness in the plastic hinge region of a shear wall (usually located at the base of the wall). According to CSA S304.1-04, the clear (unsupported) height (h) limits for standard concrete block walls (190 mm nominal thickness) are as follows:

1. Limited ductility shear walls: maximum $h = 18(190 + 10) = 3600$ mm
2. Moderately ductile shear walls: maximum $h = 14(190 + 10) = 2800$ mm
3. Moderately ductile squat shear walls: maximum $h = 20(190 + 10) = 4000$ mm

The CSA S304.1-04 height-to-thickness restrictions for limited ductility and moderately ductile shear walls must be followed and cannot be relaxed according to the current version of CSA S304.1. However, Cl.10.16.6.3 states that the h/t ratio limit for moderately ductile <u>squat</u> shear walls can be relaxed, if it can be shown for lightly loaded walls that a more slender wall is satisfactory for out-of-plane stability. A possible solution involves the provision of flanges at wall ends. However, the out-of-plane stability of the compression zone, which includes the flange and sometimes a portion of the web, must be adequate. This check is demonstrated in Example 4c (Chapter 4), where a moderately ductile squat shear wall with the h/t ratio of 33 and added flanges at its ends has been shown to satisfy the CSA S304.1 out-of-plane stability requirement.

The following analysis presents one method of checking if the flanged wall provides sufficient stiffness to prevent out-of-plane instability. For the purpose of this check, a wall can be considered as lightly loaded when the compressive stress f_c, due to the dead load (corresponding to the axial load, P_{DL}), is less than $0.1f'_m$, that is,

$$f_c = \frac{P_{DL}}{l_w t} < 0.1 f'_m$$

Consider a wall section with flanges added at both ends to enhance the out-of-plane stability shown in Figure 2-27a. The wall is subjected to the factored axial load P_f, the bending moment M_f, and is reinforced with both concentrated reinforcement of area A_c, at each end, and distributed reinforcement along the wall length (total area A_d).

The effective flange width, b_f, can be initially estimated, and then revised if the out-of-plane stability is not satisfactory. A good initial minimum estimate would be

$$b_f \approx 2t$$

where t denotes the wall thickness (see Figure 2-27b). Note that this is an iterative procedure and the flange width may need to be increased to satisfy the stability requirements.

The buckling resistance of the compression zone should be checked according to the procedure described below.

First, the area of the compression zone A_L can be determined from the equilibrium of vertical forces shown in Figure 2-27a:

$$P_f + T_1 + T_2 - C_3 - C_m = 0$$

where

$$T_1 = C_3 = \phi_s f_y A_c$$

$$T_2 = \phi_s f_y A_d$$

$$C_m = \left(0.85\phi_m f'_m\right) A_L$$

thus

$$A_L = \frac{P_f + \phi_s f_y A_d}{0.85\phi_m f'_m}$$

The area of the compression zone can be determined from the geometry shown in Figure 2-27b, that is,

$$A_L = a * t + (b_f - t) * t$$

Thus, the depth of the compression zone a can be found from the above equation as follows

$$a = \frac{A_L - b_f * t + t^2}{t}$$

The distance from the centroid of the masonry compression zone to the extreme compression fibre is equal to

$$x = \frac{t*\left(a^2/2\right)+(b_f - t)\left(t^2/2\right)}{A_L}$$

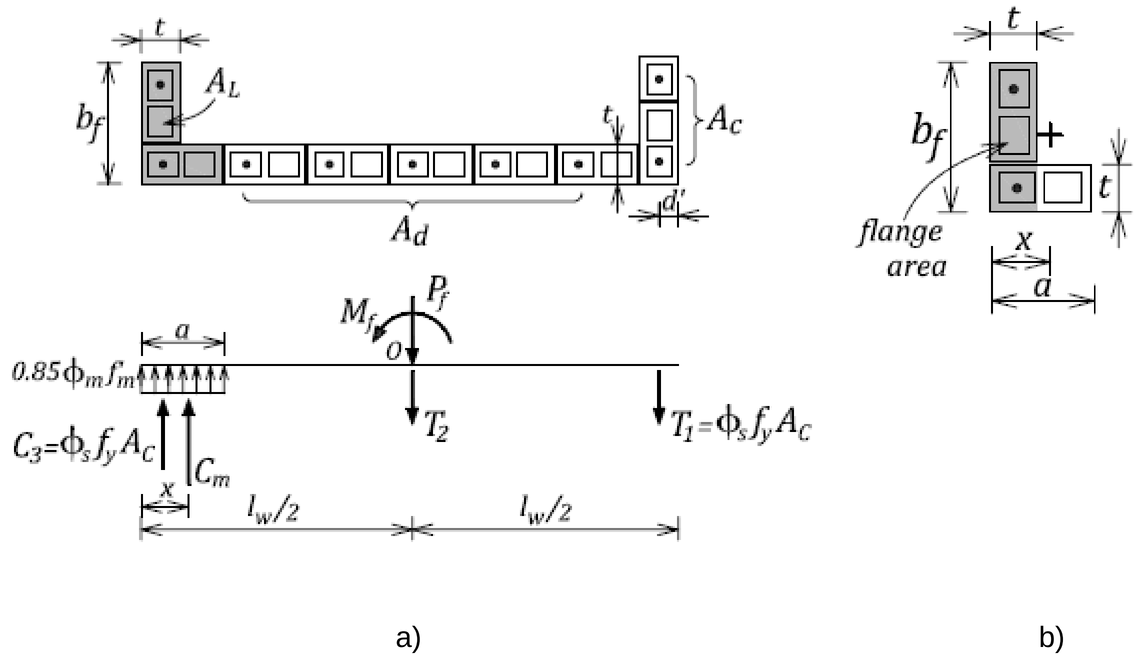


Figure 2-27. Flanged wall section: a) internal force distribution; b) flange geometry.

The compression zone of the wall may be either L-shaped or rectangular (non-flanged), however only the flange area will be considered for the buckling resistance check (the flange area is shown shaded in Figure 2-27b). This is a conservative approximation and it is considered to be appropriate for this purpose. The gross moment of inertia of the flange section around the axis parallel with the logitudinal wall axis can be determined from the following equation

$$I_{xg} = \frac{t*b_f^{\,3}}{12}$$

The use of gross moment of inertia, as opposed of partially or fully cracked one, is considered appropriate in this case, because the web portion of the compression zone and the effect of the reinforcement have been ignored.

The buckling strength for the compression zone will be determined according to S304.1 Cl. 10.7.4.3, as follows:

$$P_{cr} = \frac{\pi^2 \phi_{er} E_m I}{\left(1+0.5\beta_d\right)\left(kh\right)^2}$$

where

$\phi_{er} = 0.75$ resistance factor for member stiffness

$k = 1.0$ effective length factor for compression members (equal to 1.0 for pin-pin support conditions – a conservative assumption which can be used for this application)

$\beta_d = 0$ ratio of factored dead load moment to total factored moment (equal to 0 when 100% live load is assumed)

E_m - modulus of elasticity for masonry

The resultant compression force, including the concrete and steel component, can be determined as follows:

$$P_{fb} = C_m + \phi_s f_y A_c$$

The out-of-plane buckling resistance is considered to be adequate when

$$P_{fb} < P_{cr}$$

In some cases, it is not possible to use flanged wall sections due to architectural or other constraints. In such cases, structural designers may consider the following recommendations regarding the thickness restrictions for moderately ductile squat shear walls:

- When the CSA S304.1 h/t limits for ductile walls spanning in vertical direction (i.e. between adjacent floor slabs) have not been met, vertical supports in the form of pilasters can be provided to overcome this restriction. Clear span between adjacent pilasters should remain within the CSA S304.1 prescribed h/t limits. For more details related to the pilaster design refer to Drysdale and Hamid (2005) and Hatzinikolas and Korany (2005).
- The New Zealand Masonry Standard NZS4230:2004 (Cl.7.4.4.1) allows some relaxation for h/t ratio provided that c/t and c/l_w ratios are within certain limits. For shear walls of rectangular cross section shown in Figure 2-28a, the neutral axis depth needs to meet one of the following requirements (see Figure 2-28b):

  $$c \le 4t$$

  or

  $$c \le 0.3 l_w$$

  For flanged shear walls the neutral axis depth needs to meet the following requirement (see Figure 2-28c):

  $$c \le 6t$$

  where $6t$ is the distance from the inside of a wall return of minimum length $0.2h$. Note that, in the case of a flanged wall section such as that shown in Figure 2-28c, the non-flanged wall end is more critical for out-of-plane instability (for more details refer to Section 2.6.6). This check gives conservative results, as shown in Example 5b in Chapter 4.

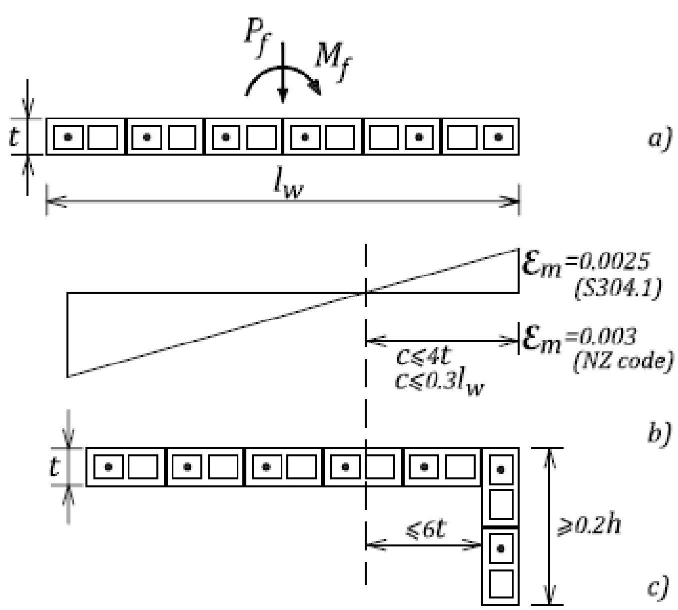


Figure 2-28. Neutral axis depth in ductile shear walls: a) rectangular (non-flanged) wall cross-section; b) corresponding neutral axis depth limits; c) flanged wall.

Note that CSA S304.1-04 restricts the maximum compressive strain in masonry ε_m in the plastic hinge zone to 0.0025, while NZS4230:2004 permits a larger strain value of 0.003.

2.5.4.5 Shear/diagonal tension resistance – seismic design requirements

10.16.4.2.1 10.16.5.3.1 10.16.6.4

CSA S304.1-04 general design provisions for shear (diagonal tension) resistance contained in Cl.10.10.1 were discussed in Section 2.3.2. Special seismic design provisions for the plastic hinge zone of the walls are as follows:

1. Limited ductility shear walls (Cl.10.16.4.2.1):
   $V_r = V_m + V_s$
   (the same equation used for the non-seismic design)

2. Moderately ductile shear walls (Cl.10.16.5.3.1):
   $V_r = 0.5V_m + V_s$
   (50% reduction in the masonry shear resistance)

3. Moderately ductile squat shear walls (Cl.10.16.6.4 and 10.10.1.3):
   $V_r = V_m + V_s$
   (the same equation used for the non-seismic design of squat shear walls)

For moderately ductile squat shear walls, Cl.10.16.6.2 requires that the shear force be applied along the entire wall length, and not concentrated near one end. The purpose of this provision is to ensure that a top transfer beam, or an alternative provision (bond beam provided at the top of the wall), will enable the development of the desirable shear failure mechanism shown in Figure 2-16a, and prevent the partial shear failure shown in Figure 2-16b. Shear failure mechanisms for squat shear walls were discussed in Section 2.3.2.2.

Commentary

Tests have shown that shear walls that fail in shear have a very poor cyclic response and demonstrate a sudden loss of strength. Also, walls that initially yield in flexure may fail in shear after several large inelastic cycles, with the resulting rapid strength degradation. Therefore, the shear steel (horizontal reinforcement) is usually designed to carry the entire shear load in the plastic hinge region of a wall (Anderson and Priestley, 1992). Seismic design provisions for ductile reinforced concrete shear walls (CSA A23.3 Cl.21.6.9) completely neglect the concrete contribution to the wall shear resistance in the plastic hinge zone.

CSA S304.1-04 provisions permit the use of the entire masonry shear resistance for all wall classes, except for moderately ductile shear walls, where only 50% of the masonry shear resistance, V_m, can be considered. CSA S304.1-94 Cl.A6.1. also contained the same reduction in the masonry shear resistance contribution for nominally ductile shear walls.

The overall shear strength is assumed to decrease in a linear fashion as the displacement ductility ratio increases, as discussed by Priestley, Verma, and Xiao (1994). This concept is illustrated in Figure 2-29 (note that displacement ductility ratio μ corresponds to the ductility-related force modification factor R_d). A ductile flexural response is ensured if the lateral force corresponding to the flexural strength is less than the residual shear strength, $V_{residual}$. A brittle shear failure takes place when the lateral force corresponding to flexural strength is greater than the initial shear strength, $V_{initial}$. When the lateral force corresponding to flexural strength is between the initial and residual shear strength, then shear failure occurs at a ductility corresponding to the intersection of the lateral force and shear force-displacement ductility plot. Anderson and Priestley (1992) recommended to allow 100% of the masonry shear strength up to ductility ratio of 2, and then to linearly decrease the masonry component of the shear strength to zero at the ductility ratio of 4. Note that CSA S304.1-04 allows 100 % of V_m up to $R_d = 1.5$, which corresponds roughly to a displacement ductility ratio of 1.5, but reduces the V_m contribution to 50 % at $R_d = 2.0$.

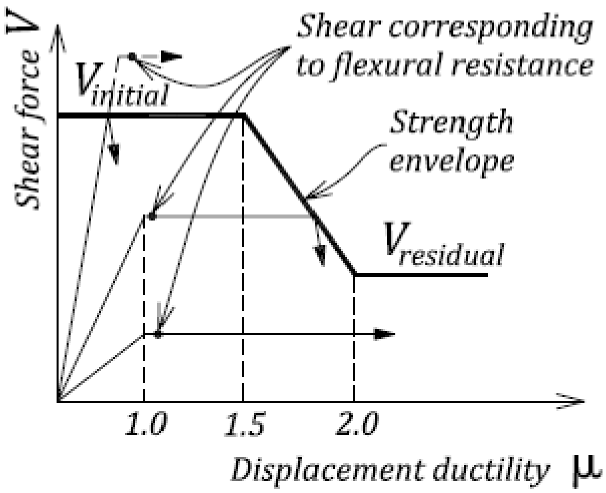


Figure 2-29. Interaction between the shear resistance and the displacement ductility ratio (adapted from Priestley, Verma, and Xiao, 1994, reproduced by permission of the ASCE[1]).

2.5.4.6 Sliding shear resistance – seismic design requirements

10.16.4.2.2
10.16.5.3.2
10.16.6.5

CSA S304.1-04 general design provisions for sliding shear resistance in Cl.10.10.4 were discussed in Section 2.3.3. The special seismic design provisions for sliding shear resistance are as follows:

1. *Limited ductility shear walls (Cl.10.16.4.2.2) and moderately ductile squat shear walls (Cl.10.16.6.5):*
   $V_r = \phi_m \mu P_2$
   The same equation is used for non-seismic design.
2. *Moderately ductile shear walls (Cl.10.16.5.3.2):*
   In this case, only the reinforcement in the tension zone should be used for the P_2 calculation. (The compressive reinforcement is assumed to have yielded in tension in a

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previous loading cycle and is now exerting a compressive force across the shear plane as it yields in compression.)

Commentary

The mechanism of sliding shear resistance was discussed in detail in the Commentary portion of Section 2.3.3. The sliding shear resistance mechanism for moderately ductile walls is illustrated in Figure 2-17b, and is unchanged from CSA S304.1-94 Clause 6.2.

It should be noted that sliding shear often governs the shear strength of reinforced masonry walls, particularly for squat shear walls in low-rise masonry buildings. To satisfy the sliding shear requirement, an increase in the vertical reinforcement area is often needed. However, this increases the moment capacity and the corresponding shear force required to yield the ductile flexural system, so the sliding shear requirement is not satisfied. Dowels at the wall-foundation interface can improve sliding shear capacity but they may also increase the bending capacity if they are too long. Note that, for squat shear walls it is impossible to prevent sliding shear if the shear reinforcement is designed to meet the capacity design requirements. In that case, shear keys could be used to increase the sliding shear resistance.

To minimize the chances of sliding shear failure, TCCMAR's findings recommended roughening the concrete foundation surface at the base of the wall with the roughness ranging from 1.6 mm (1/16 in) to 3.2 mm (1/8 in). A more effective solution is to provide the shear keys at the base of the wall that are as wide as the hollow cores and 38 mm (1.5 in) deep, with sides tapered 20 degrees. Tests have shown that these shear keys eliminate the wall slippage under severe loading (Wallace, Klingner, and Schuller, 1998).

2.5.4.7 Seismic reinforcement requirements

CSA S304.1-04 includes several requirements pertaining to the amount and distribution of horizontal and vertical wall reinforcement. It should be noted that shear walls with conventional construction do not require the special seismic detailing required for limited ductility and moderately ductile walls. These walls only need to be designed to resist the effect of factored loads, and to satisfy minimum seismic reinforcement requirements. Note that, according to NBCC 2005 Cl.4.1.8.1.1), seismic design needs to be considered when $S(0.2) > 0.12$. Also, it is possible to use unreinforced masonry constructions at sites where $I_E F_a S_a(0.2) < 0.35$ (S304.1-04 Cl.4.5.1). Shear walls reinforcement requirements are summarized in Table 2-2, with a reference being made to pertinent CSA S304.1 clauses.

Table 2-2. CSA S304.1-04 Wall Reinforcement Requirements: Loadbearing Walls and Shear Walls

<table>
<tr><th></th><th>Non-seismic design requirements</th><th>Additional seismic requirements for $I_E F_a S_a(0.2) \ge 0.35$</th></tr>
<tr><td rowspan="2">Minimum area: vertical & horizontal reinforcement</td><td>Clause 10.15.1.1</td><td>Clause 10.15.2.2</td></tr>
<tr><td>Minimum vertical reinforcement for loadbearing walls subjected to axial load plus bending shall be
$A_{v\min} = 0.0013A_g$ for $s \le 4t$
$A_{v\min} = 0.0013(4t^2)$ for $s > 4t$
S304.1 does not contain provisions regarding the minimum horizontal reinforcement area.</td><td>Loadbearing walls (including shear walls) shall be reinforced horizontally and vertically with steel having a minimum total area of $A_{stotal} = 0.002A_g$ distributed with a minimum area in one direction of at least $A_{v\min} = 0.00067A_g$ (approximately one-third of the total area)

Reinforcement equivalent to at least one 15M bar shall be provided around each masonry panel and around each opening exceeding 1000 mm in width or height. Such reinforcement shall be detailed to develop the yield strength of the bars at corners and splices.</td></tr>
<tr><td rowspan="2">Maximum area: vertical & horizontal reinforcement</td><td colspan="2">Clause 10.15.3</td></tr>
<tr><td colspan="2">Maximum horizontal or vertical reinforcement area
$A_{s\max} = 0.02A_g$ for $s \le 4t$
$A_{s\max} = 0.02(4t^2)$ for $s > 4t$

Maximum vertical reinforcement for flexural walls under low axial load (Cl.10.7.4.6.5)
$\frac{c}{d} \le \frac{600}{600 + f_y}$ or $\rho \le \rho_b$</td></tr>
</table>

<table>
<tr><th></th><th>Non-seismic design requirements</th><th>Additional seismic requirements for $I_E F_a S_a(0.2) \geq 0.35$</th></tr>
<tr><td rowspan="2">Spacing: vertical reinforcement</td><td>Clause 10.15.1.2</td><td>Clause 10.16.4.3.2</td></tr>
<tr><td>Where vertical reinforcement is required to resist flexural tensile stresses, it shall be
a) continuous between lateral supports;
b) spaced at not more than 2400 mm along the wall;
c) provided at each side of openings over 1200 mm long;
d) provided at each side of movement joints, and
e) provided at corners, intersections and ends of walls.</td><td>Vertical seismic reinforcement shall be uniformly distributed over the length of the wall. Its spacing shall not exceed the lesser of
a) $6(t+10)$ mm
b) 1200 mm
c) $l_w/4$ (for limited ductility or moderately ductile walls only)
but it need not be less than 600 mm</td></tr>
<tr><td rowspan="2">Spacing: horizontal reinforcement</td><td>Clause 10.15.1.3</td><td rowspan="2">Outside plastic hinge regions (Cl.10.15.2.6):
Horizontal seismic reinforcement shall be continuous between lateral supports. Its spacing shall not exceed
a) 400 mm where only joint reinforcement is used;
b) 1200 mm where only bond beams are used; or
c) 2400 mm for bond beams and 400 mm for joint reinforcement where both are used.

Plastic hinge regions (Cl. 10.16.4.3.3):
Reinforcing bars are to be used in the plastic hinge region, at a spacing not more than
a) 1200 mm or
b) $l_w/2$</td></tr>
<tr><td>Where horizontal reinforcement is required to resist effects of shear forces, it shall be:
a) continuous between lateral supports;
b) spaced not more than 2400 mm o/c for bond beam reinforcement;
c) spaced at not more than 600 mm for joint reinforcement for 50% running bond and 400 mm for other patterns;
d) provided above and below each opening over 1200 mm high; and
e) provided at the top of the wall and where the wall is connected to roof and floor assemblies.</td></tr>
</table>

Notes:
$A_g = 1000 \cdot t$ denotes gross cross-sectional area corresponding to 1 m wall length (for vertical reinforcement), or 1 m height (for horizontal reinforcement)
s = bar spacing
t = actual wall thickness
l_w = wall length

CSA S304.1-04 provisions related to detailing of reinforcement in ductile shear walls are summarized in Table 2-3.

Table 2-3. CSA S304.1 Reinforcement Detailing Requirements for Ductile Shear Walls

	Limited Ductility Shear Walls	**Moderately Ductile Shear Walls**
Vertical reinforcement	No special detailing requirements	***Clause 10.16.5.4.1.*** At any section within the *plastic hinge region*, no more than half of the area of vertical reinforcement may be lapped.
Horizontal reinforcement	***Clause 10.16.4.3.3*** Horizontal reinforcement shall not be lapped within a) 600 mm or b) c (the neutral axis depth) whichever is greater, from the end of the wall.	***Clause 10.16.5.4.2*** Horizontal reinforcement shall be: a) provided by reinforcing bars only (no joint reinforcement!); b) continuous over the length of the wall (can be lapped in the centre), and c) have 180° hooks around the vertical reinforcing bars at the ends of the wall.

CSA S304.1-04 minimum seismic reinforcement requirements for all classes of reinforced masonry shear walls are illustrated in Figure 2-30. To ensure desirable seismic performance of ductile shear walls, CSA S304.1-04 prescribes additional reinforcement requirements which are illustrated in Figure 2-31.

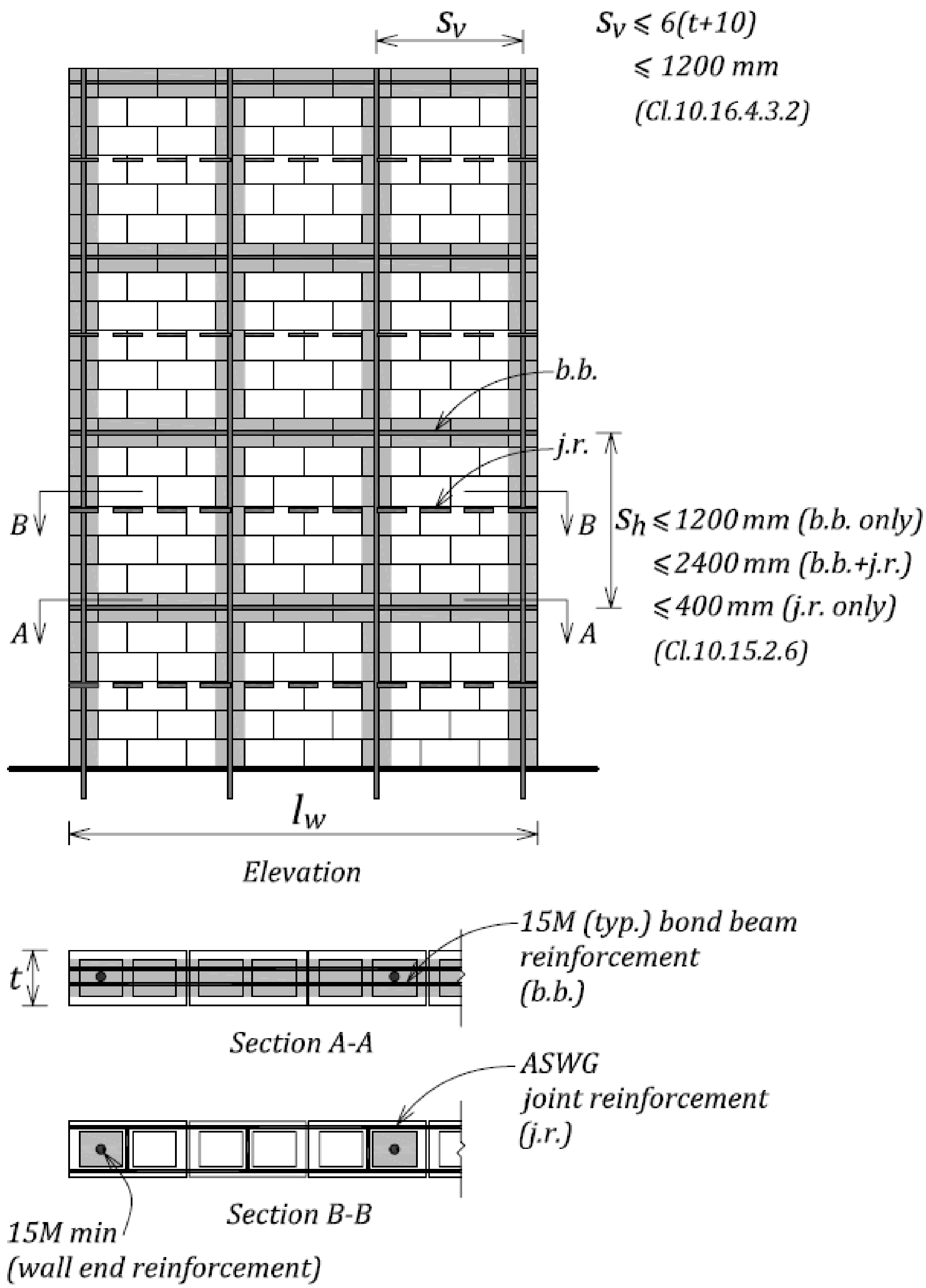


Figure 2-30. <u>Reinforced masonry shear walls:</u> CSA S304.1 minimum seismic reinforcement requirements.

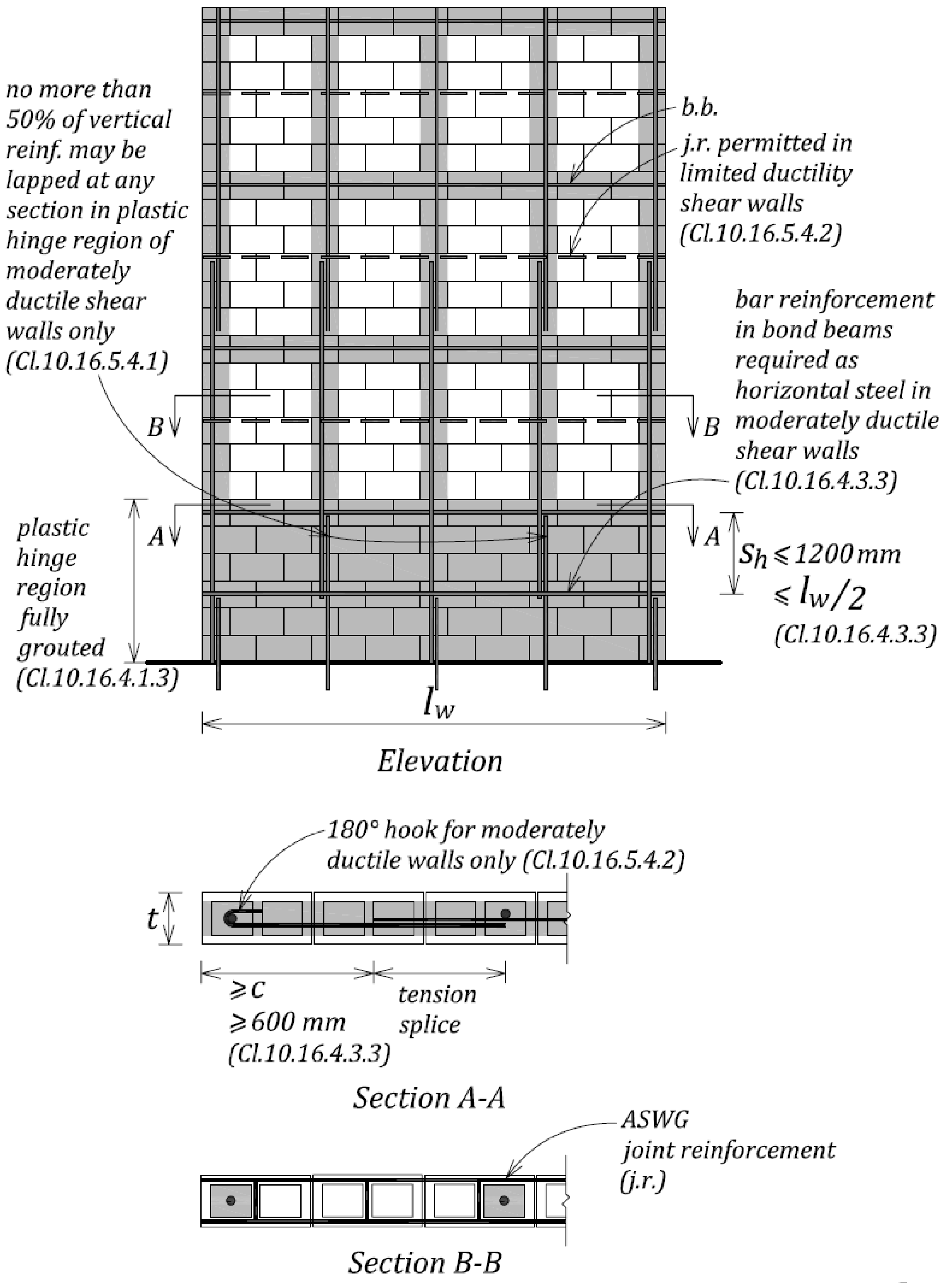


Figure 2-31. Ductile reinforced masonry shear walls: additional CSA S304.1 minimum seismic reinforcement requirements.

Commentary

According to Cl.10.16.4.3.2, vertical seismic reinforcement shall be uniformly distributed over the length of the wall. Shear walls with distributed reinforcement have almost the same moment resistance as shear walls with reinforcement concentrated at the edges, but distributed reinforcement has a beneficial effect on controlling cracking and maintaining shear strength in these walls.

Clause 10.16.4.3.3 requires that horizontal reinforcement laps not be within the greater of

- 600 mm or
- the neutral axis depth c

from the end of the wall, as shown in Figure 2-31. This requirement guards against lap splice failure in the end sections that may have either large masonry strains in the vertical direction, or masonry damage from previous cycles.

According to Cl.10.16.5.4.1, at any section within the *plastic hinge region* of moderately ductile walls, no more than half of the area of vertical reinforcement may be lapped, that is, laps should be staggered. This provision guards against failure of an entire lap splice, helps increase the hinge length, and thereby decrease the masonry strain.

Cl.10.16.5.4.2 prescribes the requirements for anchorage of horizontal shear reinforcement in moderately ductile shear walls. Adequate anchorage needs to be provided at each end of a potential diagonal crack. CSA S304.1-04 requires 180° hooks around the vertical reinforcing bars at the ends of the wall (see Figure 2-32a). Although this type of anchorage is most efficient, it may cause congestion at the end zone for narrow blocks. The New Zealand masonry design standard (NZS 4230:2004) C 10.3.2.9 recommends the use of 90° hooks bent downwards into the core as an alternative solution (see Figure 2-32b).

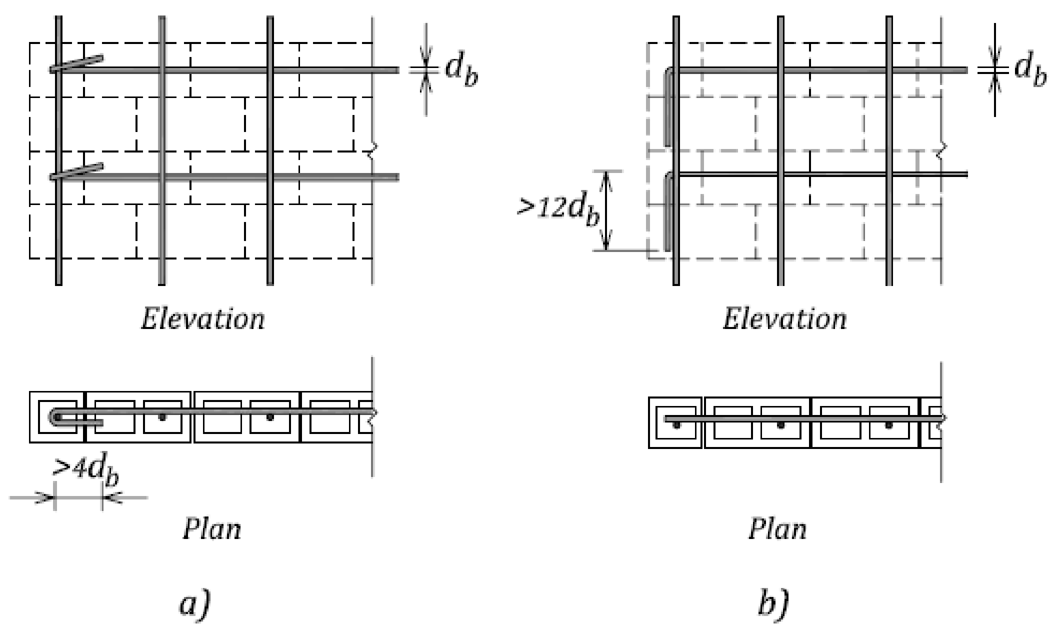


Figure 2-32. Anchorage of horizontal reinforcement: a) 180° hooks; b) 90° hooks (reproduced from NZS 4230:2004 with the permission of Standards New Zealand under Licence 000725).

CSA S304.1-94 and S304.1-04 requirements – a comparison

Most of the CSA S304.1-04 seismic requirements for shear wall reinforcement existed in the 1994 edition of the standard. A comparison is summarized below:

1. S304.1-94 Cl.5.2.2.2.2 contained the minimum seismic requirements related to reinforcement area in loadbearing walls and shear walls. These requirements remain unchanged, except that the 1994 requirements applied to the masonry structures located in seismic zone 2 or higher (this was compatible with NBCC 1995). The 2004 requirements apply to masonry structures located in areas where the seismic hazard index, $I_E F_a S_a(0.2) \geq 0.35$. This may result in changes for some locations.

   Reinforcement spacing requirements have been somewhat expanded in S304.1-04. Spacing requirements for reinforcement are clarified in Cl. 10.15.2.5 and 10.15.2.6. Cl.10.16.4.3.2 requires that the vertical reinforcement spacing for the limited ductility shear walls has an additional limit of $l_w/4$, but that it need not be less than 600 mm.

   S304.1-94 Clause A.8 included seismic reinforcement requirements for nominally ductile shear walls. The same requirements are now included in S304.1-04, Cls.10.16.4 and 10.16.5, for limited ductility and moderately ductile shear walls. A few new requirements introduced in S304.1-04 are discussed below.
2. S304.1-04 Cl.10.16.4.3.3 requires horizontal reinforcement in the form of reinforcing bars (no joint reinforcement) in the plastic hinge region of both limited ductility and moderately ductile walls (this is a new requirement).
3. S304.1-04 Cl. 10.16.5.4.2 requires 180° end hooks for horizontal reinforcement bars in the plastic hinge region of moderately ductile walls. It also requires that lines of horizontal reinforcing be continuous (this is a new requirement).

2.5.4.8 Reinforcement requirements for moderately ductile squat shear walls

10.16.6.6

CSA S304.1-04 introduced the following new requirements for the amount of reinforcement in moderately ductile squat shear walls:

- Vertical reinforcement ratio ρ_v (Cl.10.16.6.6.1):

$$\phi_s \rho_v \geq (V_f - P_f)/b_w l_w f_y$$

- Horizontal reinforcement ratio ρ_h (Cl.10.16.6.2) must be greater of

$$\phi_s \rho_h \geq \phi_s \rho_v + P_f / b_w l_w f_y$$

and that the value determined in accordance with Cl.10.10 be based on the shear resistance requirements (see Section 2.3.2).

Commentary

The seismic design requirements for moderately ductile squat shear walls were introduced in the 2004 edition of S304.1. In general, the squat wall requirements are more relaxed than those pertaining to moderately ductile flexural shear walls, because shear failure in squat shear walls is not as critical as in taller flexural walls, and can provide some ductility. Thus the design and detailing requirements related to the flexural failure mechanism (e.g. ductility check) are not required for squat walls.

The reinforcement requirements in Cl.10.16.6.6 have been derived from the mechanism of a squat shear wall failing in the shear-critical mode shown in Figure 2-33a. Consider a squat shear wall subjected to the combined effect of factored shear force, V_f, and the factored axial force, P_f. The effect of these forces can be presented in the form of distributed shear stress, v_f, and distributed axial stress, p_f, as follows

$$v_f = \frac{V_f}{b_w \cdot l_w} \qquad (14)$$

and

$$p_f = \frac{P_f}{b_w \cdot l_w} \qquad (15)$$

The wall is reinforced with horizontal and vertical reinforcement, where the reinforcement ratios ρ_h for horizontal reinforcement, and ρ_v for vertical reinforcement, are given by

$$\rho_v = \frac{A_v}{b_w \cdot l_w} \quad \text{and} \quad \rho_h = \frac{A_h}{b_w \cdot h_w}$$

where

$b_w = t$ overall wall thickness (referred to as “web width” in CSA S304.1)

l_w = wall length

h_w = wall height

If the yield stress of the reinforcement is given by f_y, the factored unit capacity of the reinforcement in the two directions is $\phi_s \rho_h f_y$ and $\phi_s \rho_v f_y$.

Once the shear force in the wall reaches a certain level, inclined shear cracks develop in the wall at a 45° angle to the horizontal axis, as shown in Figure 2-33b (note that this is an idealized model and that the angle may be different from 45°). The areas of masonry between these inclined cracks act as compression struts. Consider a typical unit length strut shown in Figure 2-33c. This strut remains in equilibrium only if there is enough force in the vertical reinforcement to satisfy moment equilibrium about the base. Note that the force in both the vertical and horizontal bars that pass through the strut do not create any net force on the strut.

Equilibrium of the strut requires that

$$p_f + \phi_s \rho_v f_y = v_f \qquad (16)$$

When v_f and p_f equations (14) and (15) are substituted into equation (16), the vertical reinforcement ratio is

$$\phi_s \rho_v = \frac{V_f - P_f}{b_w \cdot l_w \cdot f_y}$$

Note that the above equation is presented in Cl.10.16.6.6.1.

Equilibrium in the horizontal direction requires that the tensile capacity of the horizontal reinforcement, $\phi_s \rho_h f_y$, be (see Figure 2-33d)

$$\phi_s \rho_h f_y = v_f \qquad (17)$$

This equation can be presented in an alternative form useful for design purposes:

$$v_f = \frac{v_f}{\phi_s f_y} = \frac{V_f}{b_w \cdot l_w \cdot \phi_s \cdot f_y}$$

When the v_f expression is substituted from equation (17) into equation (16), it follows that

$$\phi_s \rho_h f_y = p_f + \phi_s \rho_v f_y$$

This gives the following relationship between the horizontal and vertical reinforcement, which is also presented in Cl.10.16.6.6.2:

$$\phi_s \rho_h = \phi_s \rho_v + \frac{P_f}{b_w l_w f_y}$$

It is worth noting that the required ratios of horizontal and vertical reinforcement are equal for walls with low axial load, that is, $P_f \cong 0$. This scenario applies to low-rise masonry buildings with a light roof weight.

Note that the vertical and horizontal reinforcement design should be based on flexure and shear requirements, but the designer should confirm that the minimum reinforcement requirements discussed in this section are also satisfied.

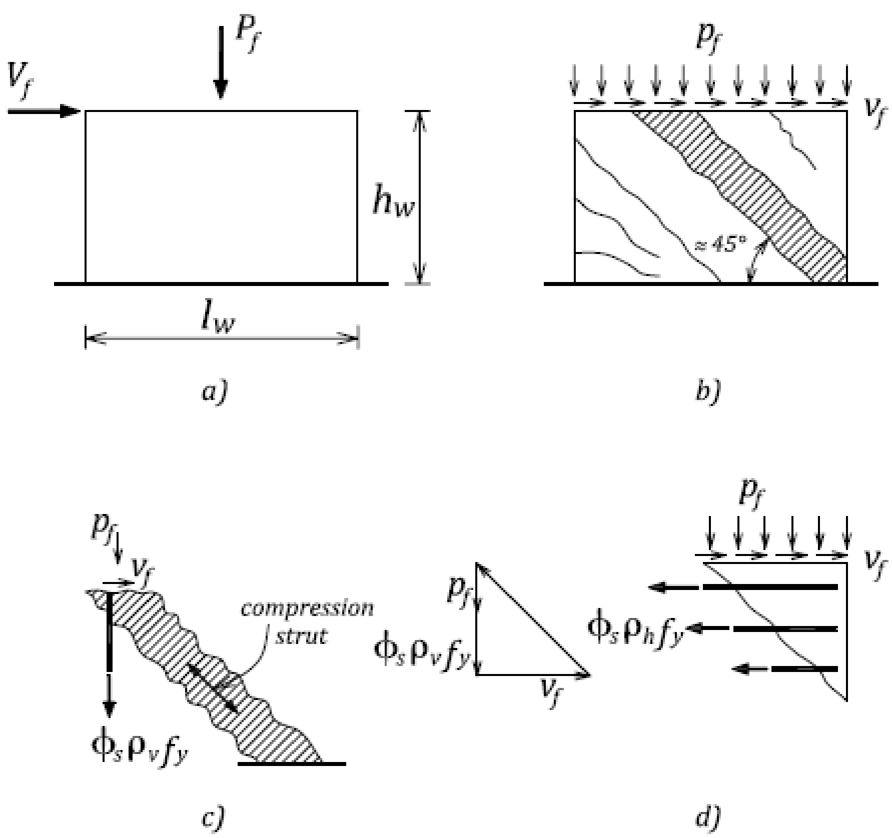


Figure 2-33. Shear failure mechanism for a squat shear wall: a) wall subjected to shear and axial load; b) crack pattern; c) compression strut; d) free-body diagram.

2.5.5 Summary of Seismic Design Requirements for Reinforced Masonry Walls

Table 2-4. Summary of the CSA S304.1-04 Seismic Design Requirements for Reinforced Masonry Walls

Provision *(guide reference section shown in the brackets)*	**Shear walls with conventional construction**	**Limited ductility shear walls**	**Moderately ductile shear walls**	**Moderately ductile <u>squat</u> shear walls ($h_w/l_w < 1$)**
Ductility factor	R_d =1.5	R_d =1.5	R_d =2.0	R_d =2.0
Plastic hinge region (2.5.4.2)	Not applicable	***Cl.10.16.4.1.1*** l_p = greater of $l_w/2$ or $h_w/6$ ***Cl.10.16.4.1.3*** Masonry within the plastic hinge region shall be fully grouted.	***Cl.10.16.5.2.1*** l_p = greater of l_w or $h_w/6$ Same as limited ductility walls	No special provisions
Ductility check (2.5.4.3)	Not applicable	***Cl.10.16.4.1.4*** 1. $\varepsilon_m = 0.0025$ 2. $c/l_w < 0.2$ when $h_w/l_w < 6$	***Cl.10.16.5.2.3*** 1. Maximum compression strain: $\varepsilon_m = 0.0025$ 2. Ductility limits: $\boxed{c/l_w < 0.2}$ when $h_w/l_w < 4$ and $\boxed{c/l_w < 0.15}$ when $4 < h_w/l_w < 8$	
Wall height-to-thickness ratio restrictions (2.5.4.4)	***Cl.10.7.3.3*** Must meet non-seismic slenderness requirements and design procedures	***Cl.10.16.4.1.2*** $h/(t+10) < 18$	***Cl.10.16.5.2.2*** $h/(t+10) < 14$	***Cl.10.16.6.3*** $h/(t+10) < 20$ Unless it can be shown for lightly loaded walls that a more slender wall is satisfactory for out-of-plane stability

Provision *(guide reference section shown in the brackets)*	**Shear walls with conventional construction**	**Limited ductility shear walls**	**Moderately ductile shear walls**	**Moderately ductile <u>squat</u> shear walls ($h_w/l_w < 1$)**
Shear/diagonal tension resistance (2.5.4.5)	***Cl.10.10.1*** $V_r = V_m + V_s$ Same as non-seismic design	***Cl.10.16.4.2.1*** $V_r = V_m + V_s$ Same as non-seismic design	***Cl.10.16.5.3.1*** $V_r = 0.5V_m + V_s$ 50% reduction in the masonry shear resistance	***Cl.10.16.6.4*** Same as limited ductility walls ***Cl.10.16.6.2*** Shear force applied uniformly along the wall length
Sliding shear resistance (2.5.4.6)	***Cl.10.10.4*** $V_r = \phi_m \mu P_2$ Same as non-seismic design	***Cl.10.16.4.2.2*** $V_r = \phi_m \mu P_2$ Same as non-seismic design	***Cl.10.16.5.3.2*** $V_r = \phi_m \mu P_2$ Only reinforcement in the tension zone to be taken into account for P_2 calculation.	***Cl.10.16.6.5*** Same as limited ductility walls
Minimum seismic reinforcement area (2.5.4.7)	Minimum seismic reinf. requirements (Cl.10.15.2.2) apply when $I_E F_a S_a(0.2) \geq 0.35$ otherwise apply minimum non-seismic reinf. requirements (Cl.10.15.1.1)	***Cl.10.15.2.2*** Minimum seismic reinforcement area requirements apply for all classes of ductile masonry walls (see *Table 2-2*)		***Cl.10.16.6.6*** Additional reinforcement requirements

2.6 Special Topics

2.6.1 Unreinforced Masonry Shear Walls

S304.1-04 allows the use of unreinforced masonry construction for sites where the seismic hazard index, $I_E F_a S_a(0.2) < 0.35$ (Cl.4.5.1). Seismic design provisions for unreinforced masonry shear walls are presented in this section.

2.6.1.1 Shear/diagonal tension resistance (in-plane and out-of-plane)

7.10.1 7.10.2

Design provisions for factored in-plane and out-of-plane diagonal tension shear resistance, V_r, for unreinforced masonry shear walls are the same as those for reinforced masonry walls, except that there is no steel contribution term ($V_s = 0$). The background for these provisions is discussed in detail in Sections 2.3.2 and 2.4.2.

Commentary

Diagonal tension is a brittle failure mode, characterized by the development of a major diagonal crack that forms when the masonry tensile resistance has been reached (see Section 2.3.1.2). This is an undesirable failure mechanism and should be avoided, preferably by providing horizontal reinforcement in masonry walls loaded in-plane and located in regions where $I_E F_a S_a(0.2) > 0.35$.

2.6.1.2 Sliding shear resistance (in-plane and out-of-plane)

7.10.4.1 7.10.4.2

Design provisions for in-plane and out-of-plane sliding shear resistance for unreinforced masonry walls are somewhat different from those for reinforced masonry, in that bed-joint sliding masonry resistance (in addition to the frictional resistance) is assigned to the wall. Note that in reinforced masonry walls only frictional resistance is considered, as discussed in Section 2.3.3.

The in-plane sliding shear resistance, V_r, along bed joints between courses of masonry, also known as *bed-joint sliding resistance*, is given in Cl.7.10.4.1 as

$$V_r = 0.16\phi_m \sqrt{f'_m} A_{uc} + \phi_m \mu P_1$$

where

μ = the coefficient of friction
= 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane
= 0.7 for a masonry-to-smooth concrete or bare steel sliding plane
= other (when flashings reduce friction that resists sliding shear, a reduced coefficient of friction accounting for the flashing material properties should be used)

P_1 = the compressive force in masonry acting normal to the sliding plane, normally taken as P_d (equal to 0.9 times the dead load). For infill shear walls, an additional component, equal to

90% of the factored vertical component of the compressive force resulting from the diagonal strut action, should be added (see Figure 2-34c).

A_{uc} = uncracked portion of the effective cross-sectional area of the wall that provides shear bond capacity (note that both out-of-plane loads and in-plane loads can cause cracking of the masonry wall)

For the in-plane sliding shear resistance, A_{uc} should be determined as follows

$$A_{uc} = t_e \cdot d_v$$

where

t_e = effective wall thickness; t_e is equal to the sum of two face shell thicknesses for hollow walls, and to the actual wall thickness t for fully grouted walls

d_v = effective wall depth, equal to $0.8 l_w$

l_w = wall length

For the out-of-plane sliding shear resistance, A_{uc} should be determined as follows

$$A_{uc} = t_e \cdot l_w$$

The sliding shear resistance at the base of the wall (along the bed joint between the support and the first course of masonry) is equal to (see Figure 2-34b)

$$V_r = \phi_m \mu C$$

where C is compressive force in the masonry acting normal to the sliding plane, normally taken as P_d (equal to 0.9 times the dead load), since T_y =0, that is,

$$C = P_d + T_y$$

Design equations for the out-of-plane sliding resistance stated in Cl.7.10.4.2 are the same as the equations for the in-plane sliding shear resistance presented above.

Commentary

The two forms of the sliding shear failure mechanism (bed-joint sliding and base sliding), are presented in Figure 2-34 a) and b). Sliding shear failure is likely to govern the design of masonry shear walls in low-rise buildings, due to the low axial load acting on these walls (see Commentary in Section 2.5.4.6). In unreinforced masonry walls, dowels can provide the required sliding shear resistance at the base, but it should be noted that a sliding shear failure can still take place at the section at the top of the dowels, which is undesirable. However, it should be noted that the sliding shear failure mechanism is a ductile one, and has been characterized by significant lateral deformations along the failure plane in major earthquakes.

Note that in the equation for bed-joint sliding resistance, the first term represents the shear bond resistance of masonry mortar, while the second term represents the sliding shear resistance based on the Coulomb friction model. In determining the sliding shear resistance for the bed-joint sliding mechanism for seismic design of unreinforced masonry walls, the first term in the equation should be ignored if the wall cracks due to either in-plane or out-of-plane bending. If the wall remains uncracked, the second term (shear friction resistance) should not be included. The smaller of the two values should be used in the design.

For the sliding resistance at the base of the wall, sliding shear resistance is provided by the weight of the wall above and yielding of steel dowels. Note that the dowel contribution is possible only after a small shear slip at the base takes place and a horizontal crack forms at the wall-to-foundation interface.

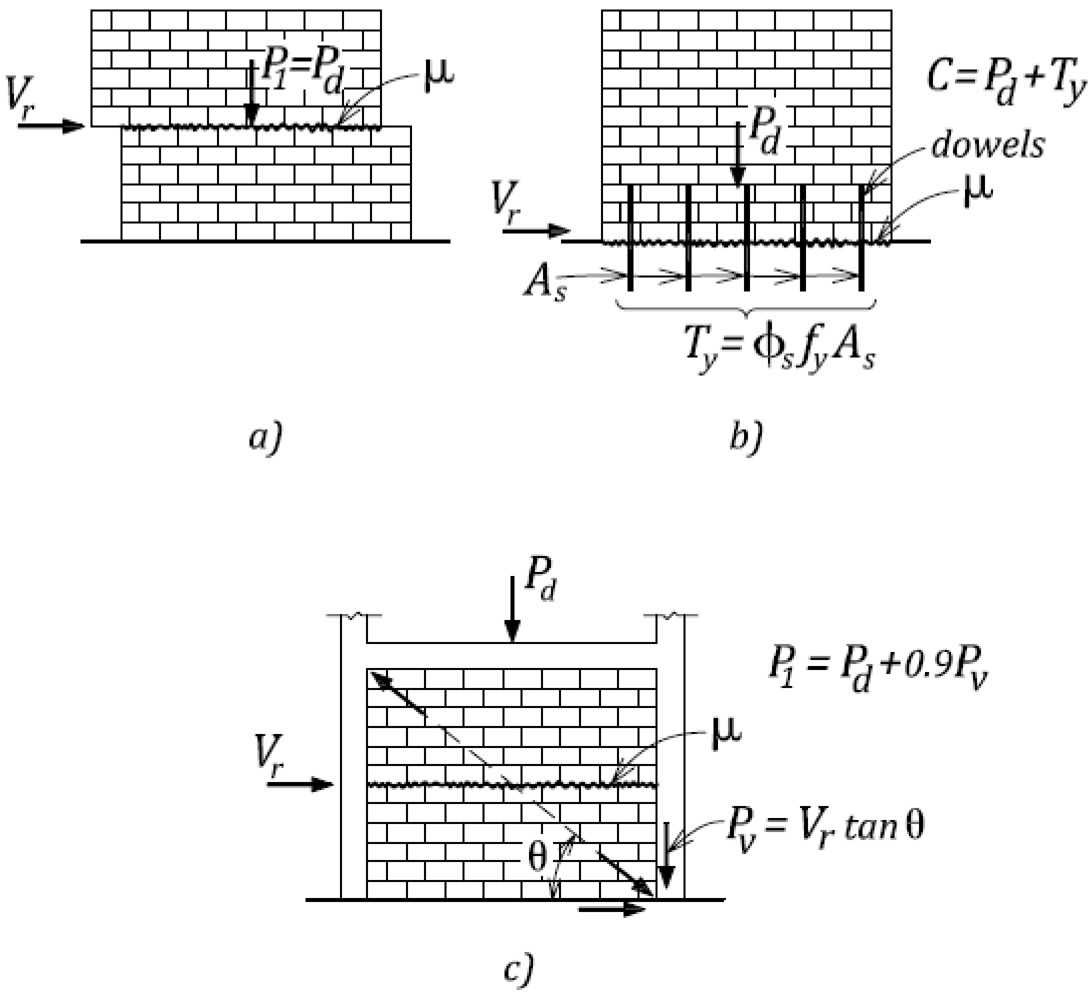


Figure 2-34. Sliding shear failure mechanism: a) bed-joint sliding; b) sliding at the base of the wall; c) sliding shear in infilled masonry walls.

The bed-joint sliding failure mechanism is also characteristic of infilled masonry walls, as shown in Figure 2-34c). Seismic design considerations for masonry infill walls are discussed in Section 2.6.2.

2.6.1.3 Flexural resistance due to combined axial load and bending

7.2

A masonry wall of length, l_w, and thickness, t, subjected to factored axial load, P_f, and factored bending moment, M_f, has an eccentricity, e, equal to

$$e = \frac{M_f}{P_f}$$

According to Cl.7.2.1, unreinforced masonry walls should be designed to remain uncracked when

$e \geq 0.33 l_w$ for in-plane bending, or

$e \geq 0.33t$ for out-of-plane bending,
but the maximum stresses must not exceed $\phi_m f_t$ for tension and $\phi_m f'_m$ for compression (Cl.7.2.2), where f_t is the flexural tensile strength of masonry (see Table 5 of CSA S304.1-04).

The maximum stresses at the wall ends can be calculated as follows:

$$\max f_c = \frac{P_f}{A_e} + \frac{M_f}{S_e} \leq \phi_m f'_m$$

and

$$\max f_t = \frac{P_f}{A_e} - \frac{M_f}{S_e} \geq -\phi_m f_t$$

where
P_f and M_f are the factored axial load and the factored bending moment acting on the wall section

$A_e = t_e \cdot l_w$ effective cross-sectional area of masonry

t_e = effective wall thickness equal to the sum of two face shell thicknesses for hollow walls, and to the actual wall thickness t for fully grouted walls

$S_e = \dfrac{t_e \cdot l_w{}^2}{6}$ section modulus of effective wall cross-sectional area

When

$e < 0.33 l_w$ for in-plane bending, or

$e < 0.33t$ for out-of-plane bending,
an unreinforced masonry wall can be designed assuming cracked wall sections (Cl.7.2.3) using an equivalent rectangular stress block, as per Cl.10.2.6.

The centroid of the compression zone must coincide with the load eccentricity, e, as shown in Figure 2-35b, and the compression capacity, P_r, can then be determined from the following equation:

$$P_r = \left(0.85 \chi \phi_m f'_m\right) \cdot t_e \cdot \left(\frac{l_w}{2} - e\right) \cdot 2$$

note that P_r must be greater than P_f.

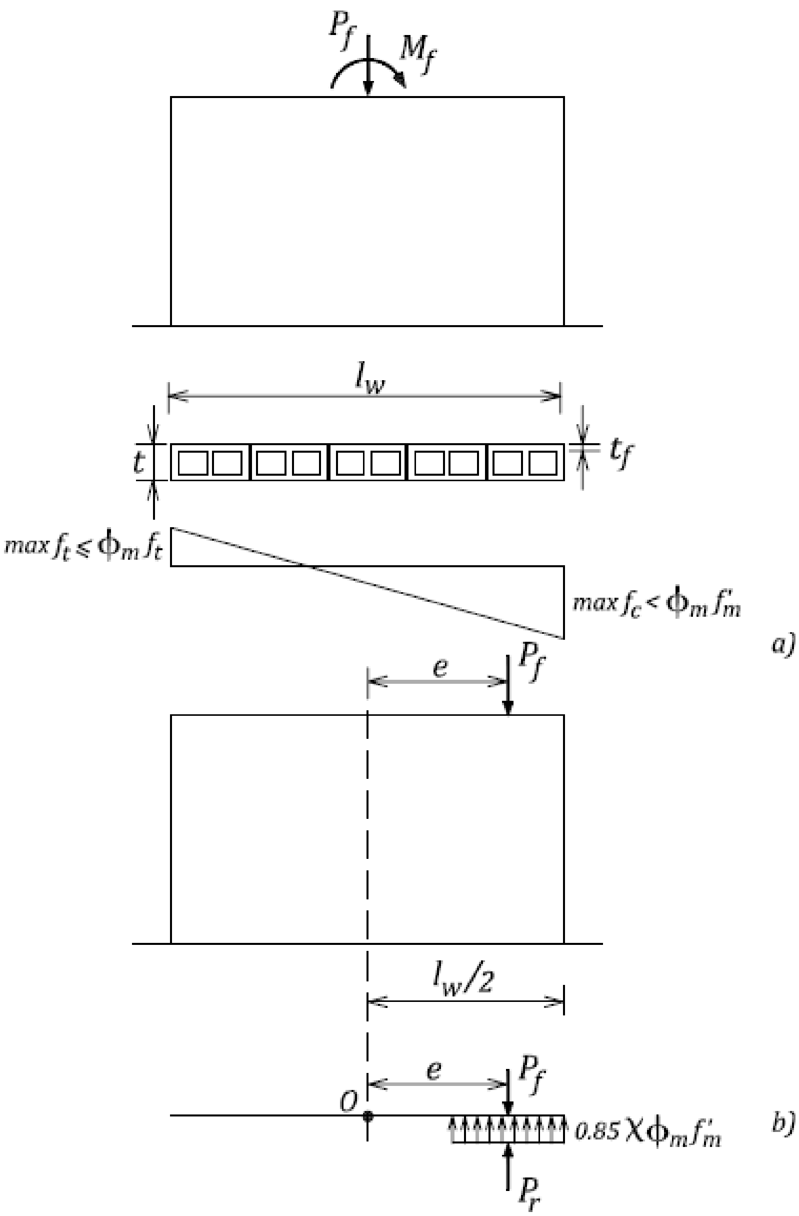


Figure 2-35. Stresses due to combined axial load and bending in an unreinforced masonry wall: a) uncracked wall; b) cracked wall.

Commentary

It is realistic to assume that unreinforced masonry wall sections will experience cracking under seismic conditions. Reports from the past earthquakes have shown that unreinforced masonry suffers extensive damage in earthquakes, e.g. 1994 Northridge, California earthquake (magnitude 6.7); for more details refer to TMS (1994). Despite the extensive damage, it should be noted that the building stock of unreinforced masonry block walls in California is very limited, since the provision for reinforcement in masonry structures started after the 1933 Long Beach earthquake. This cannot be said for most seismic zones in Canada.

2.6.2 Masonry Infill Walls

7.13 10.12

Infill walls are masonry wall panels enclosed by reinforced concrete or steel frame members on all four sides. Infill walls are not listed as a wall class in NBCC 2005, and therefore fall under the classification of “other masonry SFRS(s)”,. They are only allowed in low seismic regions where $I_E F_a S_a(0.2) < 0.20$, and have $R_d = R_o = 1.0$ and a height limitation of 15 m.

CSA S304.1 design provisions for masonry infill walls, introduced for the first time in the 2004 edition of the code, are summarized below.

<u>General design requirements</u>

1. Masonry infill walls are treated as shear walls and should be designed to resist all in-plane and out-of-plane loads (Cl.7.13.1).
2. Masonry infill walls should be designed to resist any vertical loads transferred to them by the frame (Cl.7.13.2.4).
3. The increased stiffness of lateral load-resisting elements that consist of masonry infill shear walls working with the surrounding frame, should be taken into account when distributing the applied loads to these elements (Cl.7.13.2.5).
4. When a diagonal strut is used to model the infill shear wall according to Cl.7.13.3, an infill frame can be designed using a truss model (see the note to Cl.7.13.2.5).

<u>Design approaches for masonry infill walls</u>

CSA S304.1 offers three possible design and construction approaches for infill walls:

1. *Participating infill (diagonal strut approach)* – when there are no openings or gaps between the masonry infill and the surrounding frame, but the infill is not tied or bonded to the frame, the infill should be modelled as a diagonal strut according to Cl.7.13.3. Where openings or gaps exist, the designer must show through experimental testing or special investigations that the diagonal strut action can be formed and all other structural requirements for the infill shear walls can be developed (Cl.7.13.2.3).
2. *Frame and infill composite action* – when the infill shear wall is tied and bonded to the frame to create a composite shear wall, where the infill forms the web and the columns of the frame form the flanges of the shear wall (Cl.7.13.2.2).
3. *Isolated infill* - it is also possible to design an isolated infill panel (a note to Cl.7.13.1 and Cl.7.13.2.3), which is separated from the frame structure by a gap created by vertical movement joints along the ends and a horizontal movement joint under the floor above or beam. In that case, masonry infill is a nonloadbearing wall and cannot be treated as a shear wall. Restraints must be provided at the top of the wall to ensure stability for out-of-plane seismic forces.

<u>Diagonal strut model</u>

For structural design purposes, infill walls should be modelled as diagonal struts, as shown in Figure 2-36 (Cl.7.13.2.1). The key properties of the diagonal strut model are summarized below.

Diagonal strut width w should be determined as follows (Cl.7.13.3.2):

$$w = \sqrt{\alpha_h^{\ 2} + \alpha_L^{\ 2}}$$

where

$$\alpha_h = \frac{\pi}{2}\left(\frac{4E_f I_c h}{E_m t_e \sin 2\theta}\right)^{1/4}$$

and

$$\alpha_L = \pi\left(\frac{4E_f I_b l}{E_m t_e \sin 2\theta}\right)^{1/4}$$

α_h = vertical contact length between the frame and the diagonal strut

α_L = horizontal contact length between the frame and the diagonal strut

E_m, E_f = moduli of elasticity of the masonry wall and frame material, respectively

h, l = height and length of the infill wall, respectively

$l_s = \sqrt{h^2 + l^2}$ length of the diagonal strut

t_e = sum of the thickness of the two face shells for hollow or semi-solid block units and the thickness of the wall for solid or fully grouted hollow or semi-solid block units

I_c, I_b = moments of inertia of the column and the beam of the frame respectively

θ = angle of diagonal strut measured from the horizontal, where

$$\tan\theta = \frac{h}{l}$$

Effective diagonal strut width, w_e, to be used for the calculation of the compressive strength of the strut should be taken as (Cl.7.13.3.3)

$$w_e = w/2$$

or

$$w_e \le l_s/4$$

whichever is the least.

The *design length* of the diagonal strut l_d should be equal to (Cl.7.13.3.5)

$$l_d = l_s - w/2$$

Depending on the strut end conditions (fixed or pinned), an effective length can be calculated by multiplying the design length by the effective length factor for compression members, k (see Annex B to CSA S304.1).

The design length for the diagonal strut in reinforced infill walls should be determined as the smallest of the following (Cl.10.12.3):

- design length l_d as defined above, or
- infill wall height h or length l, when minimum reinforcement and lateral anchorage are provided for the span in that direction.

<u>In-plane resistance of masonry infill walls</u>

According to CSA S304.1, masonry infills should be designed considering the following failure mechanisms:

- Compression or buckling failure in diagonal strut, and
- In-plane shear failure of the masonry infill.

Diagonal strut – compression resistance (Cl.7.13.3.4)

The compression strength of the diagonal strut, P_r, is equal to the compression strength of the masonry times the effective cross-sectional area, that is,

$P_r = (0.85 \chi \phi_m f'_m) \cdot A_e$

where

$A_e = w_e * t_e$

Note that the masonry compressive strength should be reduced by $\chi = 0.5$ (corresponding to the masonry strength for compression normal to the head joints). The concept of effective cross-sectional area is addressed by S304.1-04 Cl.7.3 (unreinforced masonry walls) and Cl.10.3 (reinforced masonry walls).

Diagonal strut – buckling resistance

In determining the compression resistance, P_r, slenderness effects should be included in accordance with Cl.7.7.

The designer should ensure that the horizontal component of the diagonal strut compression resistance, P_h, is larger than the factored shear load, V_f, acting on the infill (see Figure 2-36c).

Shear resistance of infill walls (Cl.7.13.3.1 on unreinforced infills and Cl.10.12.4 on reinforced infills)

In-plane sliding shear resistance (bed-joint sliding resistance) is the key shear resistance mechanism characteristic both of unreinforced and reinforced infill walls (Cl.7.10.4). See Section 2.6.1.2 for a discussion on the bed-joint sliding mechanism.

Infill shear walls should be designed so that a bed-joint sliding shear failure is prevented (Cl.7.13.3.1). This failure mechanism can lead to a knee-braced condition that could cause a premature failure of the column in the surrounding frame, as shown in Figure 2-39 a).

The vertical component of the diagonal strut compression resistance, P_v, must be considered in determining the sliding shear resistance, as shown in Figure 2-34 c) (see Note 2 to Cl.7.13.3.1).

CSA S304.1 Cl.10.12.4 states that the reinforced masonry infills need to be designed to resist all applied shear loads in accordance with Cl.10.10.1, as they relate to the diagonal tension shear resistance discussed in Section 2.3.2 of this guide. However, it should be noted that the shear resistance corresponding to the diagonal tension cracking does not represent the limited or ultimate load condition for infill walls (see the discussion in the commentary part of this section).

<u>Reinforcement</u>

The reinforcement is required to resist tensile and shear stresses in infills (Cl.10.12.2). The minimum reinforcement requirements stated in Cl.10.15 should be followed (see Section 2.5.4.7).

<u>Effect of masonry infill on frame members (Cl.7.13.3.1)</u>

Adjacent frame members and their connections should be designed to resist additional shear forces resulting from the diagonal strut action (see Note 3 to Cl.7.13.3.1).

Commentary

The infilling of frames is associated with the construction of medium- and high-rise steel and reinforced concrete (RC) buildings, where the frames carry gravity and lateral loads, and the infills provide the building envelope and internal partitions. Historically, the frames have been engineered according to the state of the knowledge of the time, with the infill panels considered to be “nonstructural” elements (FEMA 306, 1999). However, recent damaging earthquakes in several countries (e.g. the 1999 Kocaeli earthquake in Turkey, the 2001 Bhuj earthquake in India, the 2001 Chi Chi earthquake in Taiwan, the 2003 Boumerdes earthquake in Algeria, etc.) revealed significant deficiencies and poor seismic performance of RC frame buildings with masonry infills, thereby causing significant human and economic losses (Murty, Brzev, et al. 2006).

The introduction of infills into frames changes the lateral-load transfer mechanism of the structure from a predominantly frame action to a predominantly truss action, as shown on Figure 2-37 (Kaushik, Rai, and Jain, 2006). Masonry infills in RC or steel frame buildings are usually modelled as diagonal compression struts, so an infilled frame can be modelled as a braced frame with pin connections at beam-column joints.

It should be recognized that the seismic response of infilled frames is very complex. At low level of seismic loads, the infill panels are uncracked and often cause a significant increase in the stiffness of the entire structure. In some cases, the stiffness of a RC frame with infills may be in the order of 20 times larger than that of the bare frame. At that stage, infills usually attract most of the lateral forces, but as the load increases, the infills crack and their stiffness drops. As a result, the stiffness of an infilled frame progressively decreases in each subsequent loading cycle, and more of the load is transferred to the frame. For that reason, the frames must have sufficient strength to avoid the collapse of the structure (Kaushik, Rai, and Jain, 2006). CSA S304.1 requires that the masonry infills should be able to resist the lateral seismic loads without any assistance from the frames (Cl.7.13.3.1).

To safeguard frames from being designed for very low seismic forces, some building codes require that the frame alone be designed to independently resist at least 25% of the design seismic forces, in addition to the forces caused by gravity loads. CSA S304.1 Cl.7.13.3.1 (Note 3) states that the frame members and their connections should be designed to resist additional shear forces introduced by the diagonal strut action. For example, the columns will have to resist a shear force equal to the horizontal component of the diagonal strut compression resistance, P_h (see Figure 2-36c).

The following two analytical models can be considered in the design of infilled frames (see Figure 2-37):

i) uncracked braced frame with diagonal struts; this model results in a high stiffness (corresponding to a short period) and small lateral deflections, and
ii) bare frame with cracked frame members (assuming failed infills); this model results in a low stiffness (corresponding to a long period) and large deflections.

It should be noted that the first model will give the maximum design forces, while the second one will give the maximum lateral deflections. The designer needs to consider both models in the analysis and use the most critical values for the design.

Problems associated with seismic performance of infilled frame structures arise from discontinuities of infills along the building height, and the resulting vertical stiffness discontinuity (see the discussion on irregularities in Section 1.5.10). In such infilled frames, there is a high level of forces to be resisted by the frame components. In some cases, discontinuity of infills at the ground floor level results in a soft storey mechanism, which has caused the collapse of several buildings in past earthquakes (see Figure 2-38). In developing countries, construction quality combined with inadequate detailing of RC frame components may occur, which leads to a non-ductile seismic response of these structures.

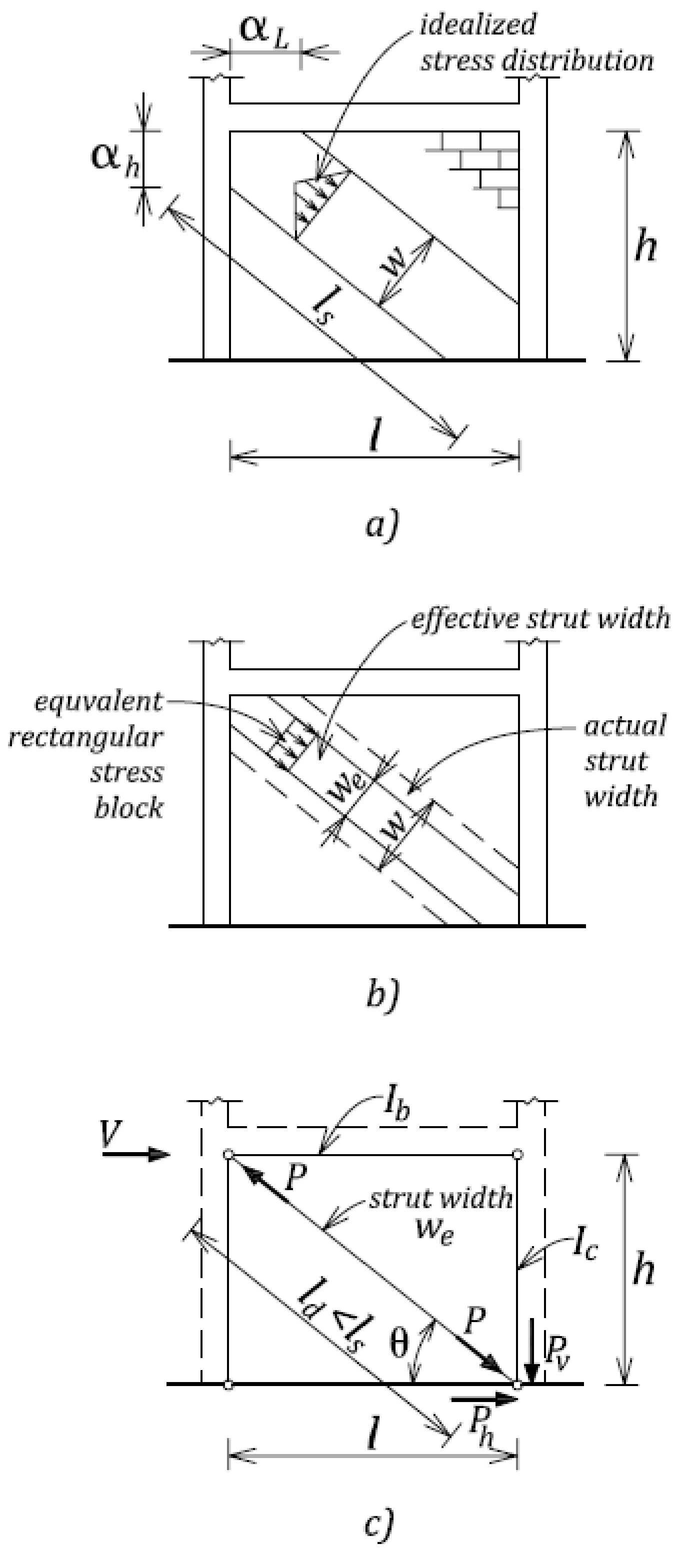
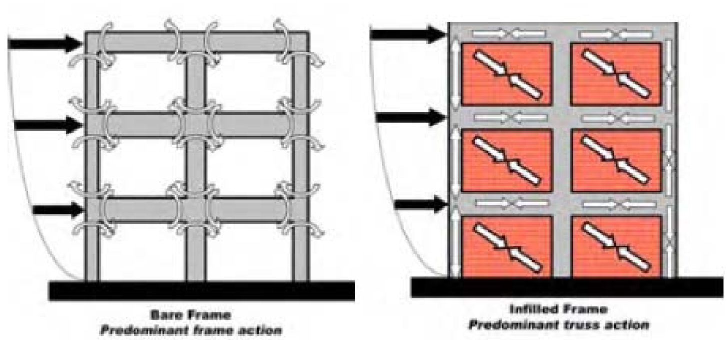


Figure 2-36. Diagonal strut model: a) actual strut width; b) effective strut width; c) analytical model.



a *b*

Figure 2-37. Masonry infills alter the seismic response of a frame structure: a) bare frame; b) diagonal strut mechanism (Source: Murty, Brzev, et al. 2006[1]).

a) *b)*

Figure 2-38. Soft storey mechanism: a) vertical discontinuity in masonry infills[2] ; b) building damage in the 2003 Boumerdes, Algeria earthquake[3].

Infill walls may fail due to the effects of *in-plane* or *out-of-plane* seismic forces. *The in-plane seismic response* of masonry infills is generally governed by shear failure mechanisms. The response depends on several factors, including the relative stiffness of the infill and frame, the material properties, and the contact between the infill and frame. The following behaviour modes are characteristic of masonry infills subjected to in-plane seismic loads (Tomazevic 1999; FEMA 306, 1999):

1. *Bed-joint sliding failure:* this mechanism takes place along horizontal mortar joints and results in the separation of infill into two or more parts (see Figure 2-39 a and b). The separated parts of the masonry infill cause free column deformations, ultimately resulting in plastic hinging in the columns. This is a ductile, displacement-controlled mechanism, since the earthquake energy is dissipated through the friction along the bed joints. This

[1] Reproduced by permission of the Earthquake Engineering Research Institute (EERI)
[2] Source: Murty, Brzev, et al., 2006, reproduced by permission of the EERI
[3] Source: S. Brzev

mechanism is likely to occur when the frame is strong and flexible. If the plane of weakness forms near the column mid-height, there is a chance for a short-column effect in the frame that can lead to a shear failure (see Figure 2-39 a). Note that when an infill panel experiences the bed-joint sliding failure, an equivalent diagonal strut may not form, so that sliding becomes the governing failure mechanism.

2. *Diagonal strut mechanism with corner compression failure*: this mechanism takes place due to the high concentration of compression stresses in the diagonal strut. The formation of a diagonal strut is preceded by diagonal tension cracking in the infill shown in Figure 2-39c. These cracks start in the centre of the infill and run parallel to the compression strut. As the load increases, the cracks propagate until they extend to the corners of the panel. When the capacity of the diagonal strut has been reached, the crushing takes place over a relatively small region (see Figure 2-39 d). The onset of diagonal shear cracking should not be considered as the limiting or ultimate load condition for infill walls, because the ultimate load is governed by either the capacity of the diagonal strut or the bed-joint sliding shear resistance.

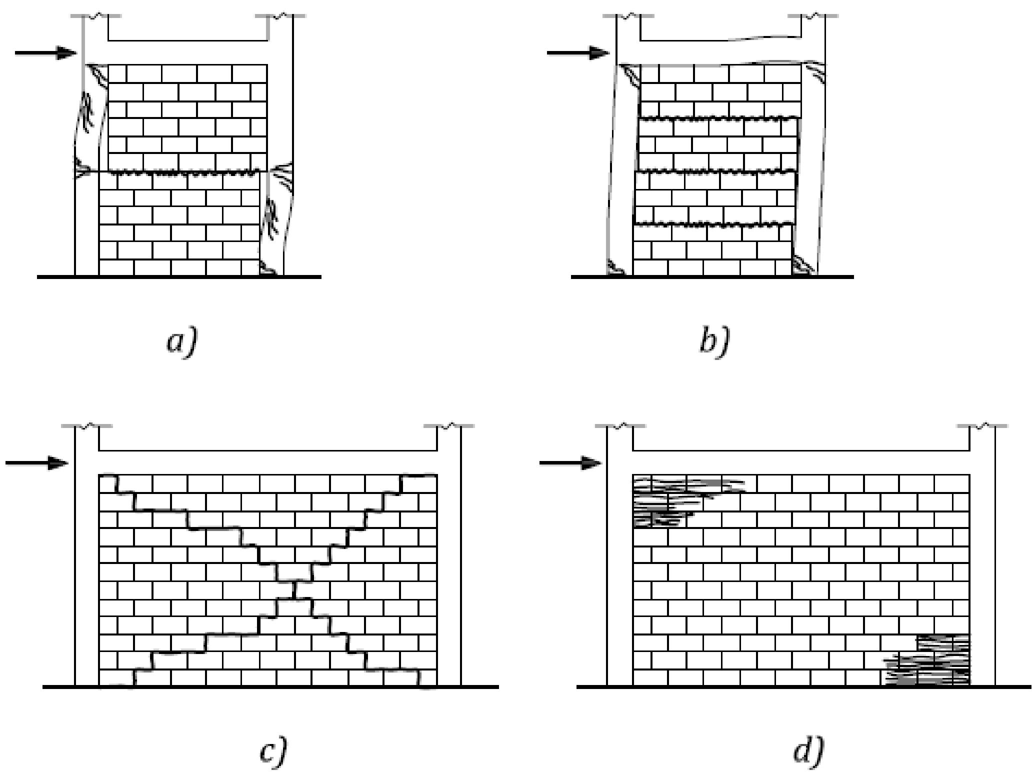


Figure 2-39. Masonry infill behaviour modes: a) and b) bed-joint sliding[1]; c) diagonal tension[2]; d) corner compression[2].

The diagonal strut mechanism can account for the additional stiffness provided by infill panels. It has been adopted by some design codes and guidelines for over 30 years, based on the pioneering research done in the1960s. It is the basis for the diagonal strut model proposed in CSA S304.1-04 (Stafford-Smith,1966), and its background has been further described in a more

[1] Tomazevic, 1999, reproduced by permission of the Imperial College Press
[2] FEMA 306, 1999, reproduced by permission of the Federal Emergency Management Agency

recent publication (Stafford-Smith and Coull, 1991). In this model, the effective strut width, w_e, is a function of the relative flexural stiffness of the column/beam and the infill, the height/length aspect ratio of the infill panel, the stress-strain relationship of the infill material, and the magnitude of diagonal load acting on the infill. Diagonal strut properties prescribed by international codes vary significantly (Kaushik, Rai, and Jain, 2006). For example, New Zealand Masonry Code NZS 4230:2004 prescribes that w_e should be taken as 25% of the length of the diagonal. Eurocode 8 (1988) prescribes that w_e should be taken as 15% of the diagonal length of the infill. The ACI 530-11 diagonal strut provisions are currently under development, as discussed by Henderson, Bennett, and Tucker (2007).

A key design parameter related to the diagonal strut model is the length of bearing (or contact) between the adjacent column and the infill (this parameter is denoted as α_h and α_L in CSA S304.1 Cl.7.13.3.2, for the column-infill or beam-infill contact length respectively). Experimental studies have shown that the bearing length is governed by the flexural stiffness of the column relative to the in-plane bearing stiffness of the infill. The stiffer the column, the longer the length of bearing, and the lower the compressive stresses at the interface (Stafford-Smith and Coull, 1991). This phenomenon is reflected in the CSA S304.1 equations used to determine α_h and α_L values. Note that the CSA S304.1 provisions are unique in that they prescribe two contact lengths – other codes and design recommendations use only the column contact length (corresponding to α_h in CSA S304.1).

Out-of-plane failure takes place due to ground shaking transverse to the plane of the wall. This mode of failure is more likely to occur at upper stories of a building, due to amplified accelerations, but it can also happen at lower stories due to concurrent in-plane loading that may damage the masonry. Arching is the prevalent mechanism in resisting out-of-plane seismic loads, because considerable out-of-plane strength can be developed even in cracked infills. This has been confirmed by several experimental studies (Dawe and Seah, 1989, and Abrams, Angel, and Uzarski, 1996). Note that the arching action is possible only for infills in direct contact with the frame (i.e. without a gap at the top). Out-of-plane strength estimates based on the flexural model of the infill acting as a vertical beam subjected to uniform load due to out-of-plane seismic load are rather conservative. Note that CSA S304.1-04 does not contain provisions related to out-of-plane resistance of masonry infills. Proposed ACI 530-11 design provisions for infill walls (currently under development), include an empirical design equation for the out-of-plane resistance of masonry infills based on the arching action, as proposed by Dawe and Seah (1989).

Isolated infill: when an infill panel is isolated from the frame, the gap (often called *seismic gap*), must be filled with a very flexible soundproof and fireproof material, e.g. boards of soft rubber or special caulking. The gap size (usually in the order of 20 to 40 mm) depends on the stiffness of the structure, the deformation sensitivity of the partition walls, and the desired seismic performance (Bachmann 2003). In addition to the gap on the sides and top of the panel, a restraint for out-of-plane resistance is required. This is typically provided in the form of clip angles or dowels at the top and/or sides that restrain out-of-plane motion only. These anchors should coincide with vertical and horizontal wall reinforcing (see CSA A370-04 for restraint information).

The above discussion mainly pertains to solid infills. Perforations within infill panels are the most significant parameter affecting seismic behaviour of infilled systems. Openings located in the centre portion of the wall can lead to weak infill behaviour. On the other hand, partial height infills can be relatively strong. The frames are often relatively weak in column shear, and partial height infills could potentially lead to a short-column mechanism (FEMA 306, 1999).

2.6.3 Stack Pattern Walls

Stack pattern is the arrangement of masonry units in which the head joints are vertically aligned (CSA S304.1 Cl.2.2.1). Stack pattern is not recommended for walls resisting seismic loads because, unlike a running bond pattern, the wall integrity provided by overlapping units is not available. The term stack pattern is now used, rather than stack bond, to highlight the lack of bond provided by this configuration of units. Stack pattern walls can be found in existing masonry buildings throughout Canada (see Figure 2-40a), and some older walls of this type are being demolished, as shown in Figure 2-40b. These walls act as a series of individual vertical columns, and the provision of horizontal reinforcement is essential to tie them together.

a)

b)

Figure 2-40. Stack pattern walls: a) stack pattern wall in an existing masonry building[1]; b) demolished stack pattern wall[2].

CSA S304.1-04 provisions regarding stack pattern walls of relevance for the seismic design are summarized in this section. CSA S304.1-94 did not contain any specific design provisions related to stack pattern walls.

[1] Credit: Svetlana Brzev
[2] Credit: Bill McEwen

2.6.3.1 Reinforcement requirements

CSA A371-04 Cl.8.1.3

Joint reinforcement or other horizontal reinforcement is required when structural or veneer masonry is laid in stack pattern, defined as less than a 50 mm overlap of masonry units.

10.10.3

Horizontal reinforcement for in-plane shear resistance in stack pattern walls shall be spaced at
- a) maximum 800 mm for bond beam reinforcing, and
- b) maximum 400 mm for wire joint reinforcing.

10.15.1
10.15.2

Reinforced stack pattern walls need to meet the minimum horizontal and vertical reinforcement requirements for non-seismic condition contained in Cl. 10.15.1, and the additional minimum seismic requirements of Cl.10.15.2 (see Section 2.5.4.7 and Table 2-2).

Commentary

Provision of horizontal reinforcement is critical for enhancing continuity in stack pattern walls. CSA S304.1-04 permits the use of joint reinforcement spaced at 400 mm or less, in addition to the bond beam reinforcement provided at a maximum spacing of 2400 mm (Cl.10.15.1.3). Codes in other countries, e.g. the U.S. masonry code ACI 530-08 (2008) Cl.1.11 states that the horizontal reifnforcement shall be placed at a maximum spacing of 48 in. (1219 mm) on center in horizontal mortar joints or in bond beams. Commentary to Cl. 1.11 states that “the use of horizontal reinforcement to enhance continuity in stack pattern walls is generally practical only by the use of bond beams”.

Note that gross cross-sectional area A_g for minimum area of vertical reinforcement according to Cl.10.15.1.1, should be calculated based on the effective compression zone width b discussed in Section 2.6.3.3.

2.6.3.2 In-plane shear resistance

10.10.3

The maximum factored vertical in-plane shear resistance in stack pattern walls shall not exceed that corresponding to the shear friction resistance of the continuous horizontal reinforcing used to tie the wall together at the continuous head joints (see Section 2.6.3.1 for horizontal reinforcement requirements).

Shear friction resistance shall be taken as

$$V_r = \phi_m \mu C_h$$

where
μ = 0.7 is the shear friction coefficient

C_h = compressive force in the masonry acting normal to the head joint. It is normally taken as the factored tensile force at yield of the horizontal reinforcement crossing the joint. This reinforcement must be detailed to develop its yield strength on both sides of the vertical joint.

Commentary

In-plane shear resistance of stack pattern walls is less than that of walls built in running bond. There is no masonry contribution to the shear resistance, so the resistance depends exclusively on the reinforcement crossing the vertical head joint. This is similar to the treatment of shear resistance at wall intersections prescribed in Cl.7.11.4 (see Section C.2).

Shear friction resistance, V_r, is proportional to the coefficient of friction, μ, and the clamping force, C_h, acting perpendicular to the wall height, h (see Figure 2-41). C_h is equal to the sum of tensile yield forces developed in reinforcement bars of area A_b, spaced at the distance s, that is:

$$C_h = \phi_s f_y A_b \, h/s$$

Reinforcing bars providing the shear friction resistance should be distributed uniformly across the vertical joint. The bars should be long enough so that their yield strength can be developed on both sides of the joint. Note that, in theory, a sliding shear plane can form along any vertical joint in a stack pattern wall.

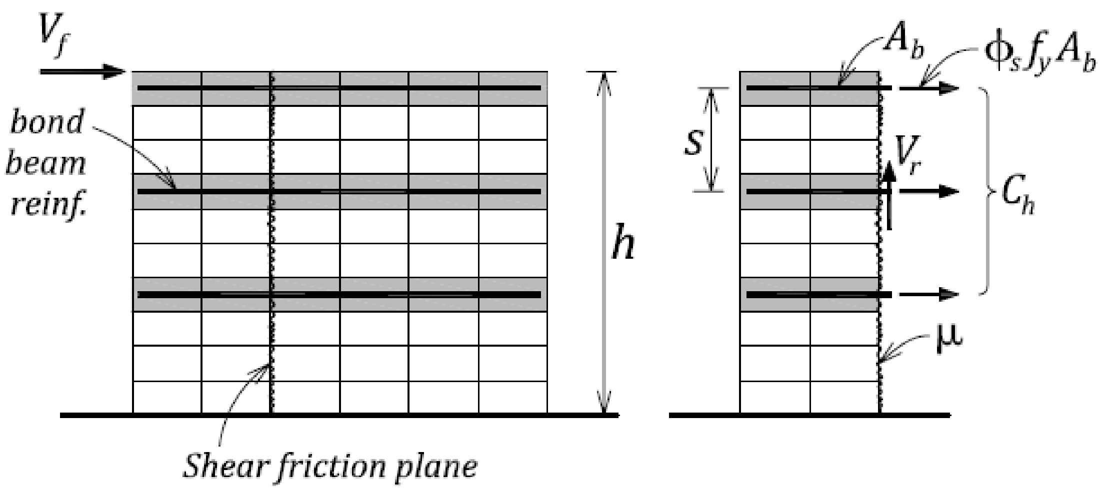


Figure 2-41. In-plane shear resistance of stack pattern walls.

2.6.3.3 Out-of-plane shear resistance

10.10.2

Out-of-plane shear resistance of stack pattern walls is determined according to the same provisions for walls built in running bond (see Section 2.4.3). Note that, for the purpose of shear resistance calculations, b includes the width of the cell and webs at a vertical bar within the length of the reinforced unit.

Commentary

Unless horizontal reinforcement is provided in sufficient amount (size and spacing), out-of-plane shear resistance of stack pattern walls is similar to that of a series of isolated vertical columns. In Figure 2-42 some stacks are not reinforced with vertical bars and so it is important to have adequate horizontal reinforcement to tie the stacks together.

2.6.3.4 Design for the combined axial load and flexure

The design approach for reinforced stack pattern walls for combined axial load and flexure is similar to that presented in Sections 2.3.4 and 2.4.4 for running bond. In determining the out-of-plane flexural resistance, the flexural tensile strength f_t should be taken equal to 0 for tensile resistance parallel to bed joints (S304.1 Cl.5.2.1). Also, the effective compression zone width b should be taken according to Cl.10.6.1.

10.6.1

For the case of out-of-plane loading (or "minor axis bending" as referred to in S304.1), the effective compression zone width, b , used with each vertical bar in the design of <u>stack pattern walls</u> with vertical reinforcement shall be taken as the <u>lesser of</u>

a) spacing between vertical bars, s , or
b) the length of the reinforced unit.

Figure 2-42 shows a portion of a reinforced stack pattern wall. In this example the length of the reinforced units is less than the spacing between bars and so the compression zone width, b , to be used with such bar is equal to the block length.

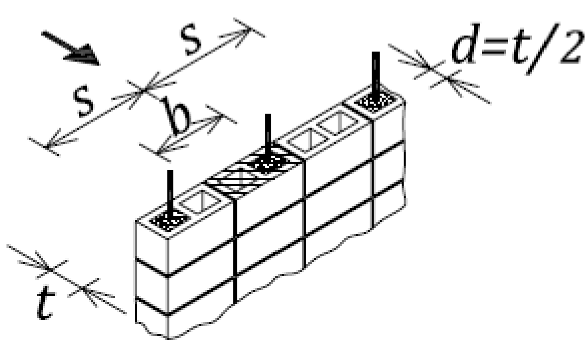


Figure 2-42. Effective compression zone width b for out-of-plane seismic effects in stack pattern walls.

2.6.3.5 Seismic requirements – plastic hinge region

10.16.4.1.3

CSA S304.1-04 permits the use of stack pattern in plastic hinge regions of ductile shear walls, however these regions must be solidly grouted and constructed of open-ended H-blocks.

Commentary

In addition to the proper amount and detailing of horizontal and vertical reinforcement in plastic hinge regions, the extent and continuity of grout is critical for the satisfactory seismic performance of reinforced masonry walls (see Section 2.5.4.2 for a detailed discussion on plastic hinge regions). Some codes, such as the New Zealand masonry code (NZS 4230:2004), do not permit the use of stack pattern walls in plastic hinge regions of the masonry walls, while the U.S. masonry code ACI 530-08 (2008) does (see Cl.1.17.3.2.6.e)..

2.6.3.6 Unreinforced stack pattern walls

CSA S304.1 does not contain any provisions related to unreinforced stack pattern walls. Cl.7.10.3 for unreinforced walls is identical to Cl.10.10.3 for the in-plane seismic resistance of reinforced stack pattern walls.

Commentary

The seismic performance of stack pattern walls without closely spaced horizontal reinforcement has been much less satisfactory than for walls constructed in running bond. The presence of horizontal reinforcement is critical for tying together vertical columns formed by stacked blocks (NZS 4230:2004).

Unreinforced stack pattern walls located in regions with moderate to high seismic risk are considered to be vulnerable to seismic effects and should be either retrofitted or demolished. It is suggested that unreinforced stack pattern walls not be used in seismic regions.

2.6.4 Nonloadbearing Walls

Nonloadbearing walls resist the effects of their own dead load and any out-of-plane wind and earthquake loads. This includes partitions and exterior walls that do not support floors and roofs (S304.1 Cl.2.2). However, walls that do not support floors and roofs, but resist the in-plane forces from wind and earthquake loads are considered loadbearing shear walls (see Section 2.5.4.7 for a detailed discussion on seismic reinforcement requirements for shear walls).

10.15.2.3
10.15.2.4

Minimum seismic reinforcement requirements for nonloadbearing walls are summarized below:

1. If $I_E F_a S_a(0.2) \leq 0.35$

   Minimum seismic reinforcement is not required per CSA S304.1-04.
2. If $0.35 \leq I_E F_a S_a(0.2) \leq 0.75$ (Cl.10.15.2.4)

   Nonloadbearing walls shall be reinforced in one or more directions with reinforcing steel having a minimum total area of

$A_{stotal} = 0.0005A_g$

The reinforcement may be placed in one direction, provided that it is located to reinforce the wall adequately against lateral loads and that it spans between lateral supports.

3. If $I_E F_a S_a(0.2) \geq 0.75$ (Cl.10.15.2.3)

   Nonloadbearing walls shall be reinforced horizontally and vertically with steel having a minimum total area of

   $A_{stotal} = 0.001A_g$ distributed with a minimum area in one direction of at least

   $A_{v\min} = 0.00033A_g$ (approximately one-third of the total area).

   A_g denotes gross cross-sectional area corresponding to unit wall length (for vertical reinforcement), or unit height (for horizontal reinforcement). Note that this minimum total area is one half of that required for loadbearing walls.

10.15.2.6

Horizontal seismic reinforcement must be continuous between lateral supports in both loadbearing and nonloadbearing walls. Its spacing cannot exceed

(a) 400 mm where only joint reinforcement is used;
(b) 1200 mm where only bond beams are used; or
(c) 2400 mm for bond beams and 400 mm for joint reinforcement where both are used.

In terms of seismic design, the effect of out-of-plane seismic loads is likely going to govern the design of nonloadbearing walls. The approach for out-of-plane flexural design is similar to that presented in Section 2.4.4 for reinforced masonry walls. For unreinforced nonloadbearing walls, the design procedure presented in Section 2.6.1.3 should be followed.

Reinforced nonloadbearing walls include masonry enclosing elevator shafts and stairways, walls used as exterior cladding (not veneers), and masonry partitions which exceed 200 kg/m^2 in mass or are over 3 m in height, and are located at the sites where $I_E F_a S_a(0.2) > 0.75$ (Cl.4.6.1). Seismic requirements for nonloadbearing walls in CSA S304.1-94 were similar to the current code requirements. Cl.6.3.3.1 stated that minimum seismic reinforcement is required for nonloadbearing walls located in velocity or acceleration-related seismic zones 2 or higher. Minimum seismic reinforcement requirements stated in Cl.5.2.2.3 and 5.2.2.4 of CSA S304.1-94 are the same as the current requirements (S304.1-04 Cl.10.15.2.3 and 10.15.2.4) in terms of reinforcement area. S304.1-94 reinforcement spacing requirements were similar to those stated in Cl.10.15.2.6 of S304.1-04. Note that the item c) in Cl.10.15.2.6 of S304.1-04 covering the case of a combination of bond beams and joint reinforcement did not exist in the previous standard.

2.6.5 Masonry Veneers and their Connections

2.6.5.1 Background

In some applications and exposure conditions, the need for better control over rain penetration led to the incorporation of an air space or cavity in traditional masonry walls to provide a capillary break between two wythes. This type of two-stage wall can be referred to as a *rainscreen wall,* when the air space behind the outermost element is drained and ventilated to the exterior and an effective air barrier is included in the backup assembly. Masonry *veneer*, an important component of a modern rainscreen wall, is a nonloadbearing masonry facing attached to, and supported laterally by a structural backing. The structural backing may be structural masonry, concrete, metal stud or wood stud. A section of a typical rainscreen wall is shown in Figure 2-43.

While masonry veneers of brick, block or stone are nonloadbearing components, there are structural issues to be addressed if they are to perform satisfactorily. Veneers must be connected adequately to a structural backing by means of metal *ties* to ensure effective transfer of lateral loads due to wind and earthquakes. Steel angles are usually used to support veneers across openings (lintels), and to provide horizontal movement joints (shelf angles). For more information related to masonry veneers refer to the Masonry Technical Manual by MIBC (2008).

Veneer design is addressed by CSA S304.1-04 Cl.9 and CSA A370-04 Connectors for Masonry.

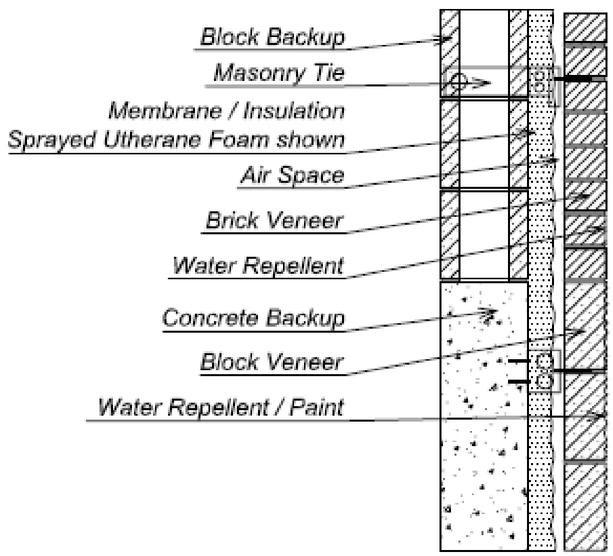


Figure 2-43. Key components of a masonry veneer (MIBC, 2008, reproduced by permission of the Masonry Institute of BC).

2.6.5.2 Ties

Ties are the key components that connect a veneer to a structural backing to ensure effective lateral load transfer. Tie requirements are outlined in CSA A370-04 Connectors for Masonry. The older kinds of ties, such as strip ties and Z-ties (now referred to as "Prescriptive Ties"), are seldom used in modern commercial construction, and cannot be used where the seismic hazard index, $I_E F_a S_a(0.2) > 0.35$. The newer, 2-piece, adjustable, engineered ties that are now in common use are now simply referred to as "Ties". CSA A370-04 contains strict design requirements for the corrosion resistance, strength, deflection and free play of ties.

CSA A370-04 requires stainless steel ties for masonry over 13 m high (formerly "buildings" over 11 m in CSA A370-94) for areas subject to high wind-driven rain. Hot dipped galvanized coatings are the acceptable minimum corrosion protection for walls 13 m or lower in these areas, and for all walls in drier areas. The standard provides wind-driven rain data for locations across Canada in Annex E, in terms of their Annual Driving Rain Index (aDRI).

The maximum tie spacing is prescribed by S304.1-04 Cl.9.1.3 and A370-04 Cl.7.1 as follows

- 600 mm vertically, and
- 820 mm horizontally

Note that S304.1-04 and A370-04 prescribe different value for horizontal tie spacing – the value of 820 mm prescribed by S304.1-04 is stated here because it better reflects construction dimensions.

While these tie spacings may be feasible for stiff backups like block and concrete, in most cases they cannot be achieved under the calculation method specified for flexible stud backups. The wind load lateral deflection limit for flexible stud backups supporting masonry veneer is span/360.

The factored resistance of a tie (P_r) is addressed by A370-04 Cl.9.4.2.1.2, and can be determined from the following equation

$P_r = \phi * P_{ult}$

where ϕ is the the resistance factor, which can assume the following values

ϕ = 0.9 for tie material strength

ϕ = 0.6 for embedment failure, failure of fasteners, or buckling failure of the connection.

P_{ult} denotes the ultimate tie strength. A370-04 requires that the ultimate strength of a masonry tie be not less than 1000 N.

2.6.5.3 Seismic load provisions for ties

Seismic load provisions for ties apply in areas in which the seismic hazard index $I_E F_a S_a(0.2) > 0.35$, and for all post-disaster buildings (NBCC 2005 Cl.4.1.8.17.2).

Ties are designed to resist the lateral wind and seismic loads acting perpendicular to the veneer surface, based on the tributary tie area. Seismic lateral loads on ties are determined from the provisions for elements and components of buildings and their connections (NBCC 2005 Cl. 4.1.8.17). The seismic tie load V_p is determined from the following equation:

$$V_p = 0.3 F_a S_a(0.2) I_E S_p W_p$$

where

- $S_a(0.2)$= 5 % damped spectral response acceleration for a 0.2 sec period (depends on the site location; values for various locations in Canada from NBCC 2005 Appendix C)
- F_a = foundation factor, which is a function of site class (soil type) and $S_a(0.2)$ (see Section 1.5.2)
- I_E = building importance factor equal to1.0, except 1.3 for schools and community centres, and 1.5 for post-disaster buildings
- $S_p = C_p A_r A_x / R_p$ (where $0.7 < S_p < 4.0$)
- S_p = horizontal force factor for part or portion of a building and its anchorage (see NBCC 2005, Table 4.1.8.17, Case 8)
- C_p = seismic coefficient for a particular nonstructural component (equal to 1.0 for ties)
- A_r = response amplification factor to account for the type of attachment (equal to 1.0 for ties)
- $A_x = 1 + 2h_x / h_n$ amplification factor to account for variation of response with the height of the building (maximum 3.0 for the worst case at top of wall for ties). Note that $A_x = 3$ is the worst case for a tall building that may have higher mode contribution to accelerations in the top part of the building; thus $A_x = 3$ would be used for the entire top floor. For a single-storey building this doesn't make much sense. However, the accelerations will be higher at the top of a wall where the capacity is reduced because of low vertical load on the bricks, so $A_x = 3$ may be reasonable for the top row of ties. This could be reduced in the lower part of the wall, but for construction simplicity it would be better to maintain one spacing on most projects.
- R_p = element or component response modification factor (equal to 1.5 for ties).

So, the S_p value for tie design is

$S_p = 1.0 \cdot 1.0 \cdot 3.0/1.5 = 2.0$

W_p = tributary weight for a specific tie, equal to the unit weight of the veneer masonry (typically taken as 1.8 kN/m^2 for brick and cored block) times the tributary area (equal to the product of tie spacing for each direction).

The tie design load depends on the type of veneer backup (rigid/flexible), as per S304.1 Cl.9.1.3.3:

- For rigid backups (e.g. concrete block walls), the tie force is equal to the seismic load V_p corresponding to the tributary area weight W_p .
- For flexible backups (e.g. steel or wood stud walls), a tie must resist 40% of the tributary lateral load on a vertical line of ties. However, a tie must also be able to resist the load from double the tributary area on the tie.

The new formula from the NBCC 2005 may result in lower lateral seismic loads than the NBCC 1995 and may result in wind loads governing in more cases.

Factored tie capacities V_r are normally provided by test data from the manufacturers. The tie capacity is considered to be adequate provided that

$$V_p \le V_r$$

If this is not a case, the tributary area and resulting tie spacing can be reduced until the above requirement is satisfied, or a stronger tie can be considered. In many cases, the design will begin with a given tie strength, with the resulting spacing calculated and assessed (see design Example 7 in Chapter 4).

2.6.6 Boundary Elements and Flanged Shear Walls

CSA S304.1-04 does not contain any specific seismic provisions regarding boundary elements in reinforced masonry shear walls. Boundary elements are thickened and specially reinforced sections provided at the ends of shear walls (see Figure 2-44a). The practice of using boundary elements is common for reinforced concrete ductile shear walls, with the related seismic design provisions included in CSA A23.3-04. In tall shear walls subjected to significant bending moments at their base, boundary elements provide an additional space to accommodate confinement and additional vertical flexural reinforcement. Boundary elements also provide stability against lateral out-of-plane buckling in thin wall sections (this was discussed in Section 2.5.4.4). To sustain high flexural and normal stresses, vertical reinforcement in the boundary elements must be well confined using properly anchored transverse reinforcement. This applies particularly to the plastic hinge regions of shear walls. Since seismic bending moments in reinforced masonry shear walls in low- and medium-rise buildings are not as high as those expected in RC shear walls in high-rise buildings, the provision of boundary elements may not be required.

It is of interest to note that U.S. masonry design standard ACI 530-08 (Clauses 3.3.6.5.1 to 3.3.6.5.5) contains provisions for boundary elements in reinforced masonry shear walls. However, Cl.3.3.6.5.1 states that it is expected that boundary elements will not be required in lightly loaded walls (e.g. $P_f \le 0.1 A_g f'_m$ for symmetrical wall sections), in walls that are either short (squat) or moderate in height (aspect ratio $M_f / V_f l_w < 1.0$), or in walls subjected to moderate shear stresses. It is expected that most masonry shear walls in low- to medium-rise buildings would not develop high enough compressive strains to warrant special confinement.

Boundary elements may be required in shear walls to satisfy height-to-thickness requirements, or in walls in which flexural failure governs with the c/l_w ratio exceeding certain limit (similar to the ductility check procedure discussed in Section 2.5.4.3). For more details refer to ACI 530-08 Cl.3.3.6.5.3 and the commentary.

Flanged shear walls are discussed in Section C.2. A typical L-shaped flanged wall section is shown in Figure 2-44b. CSA S304.1-04 does not contain any specific seismic provisions related to flanged shear walls. Flanged shear walls are required to resist earthquake forces in both principal directions.

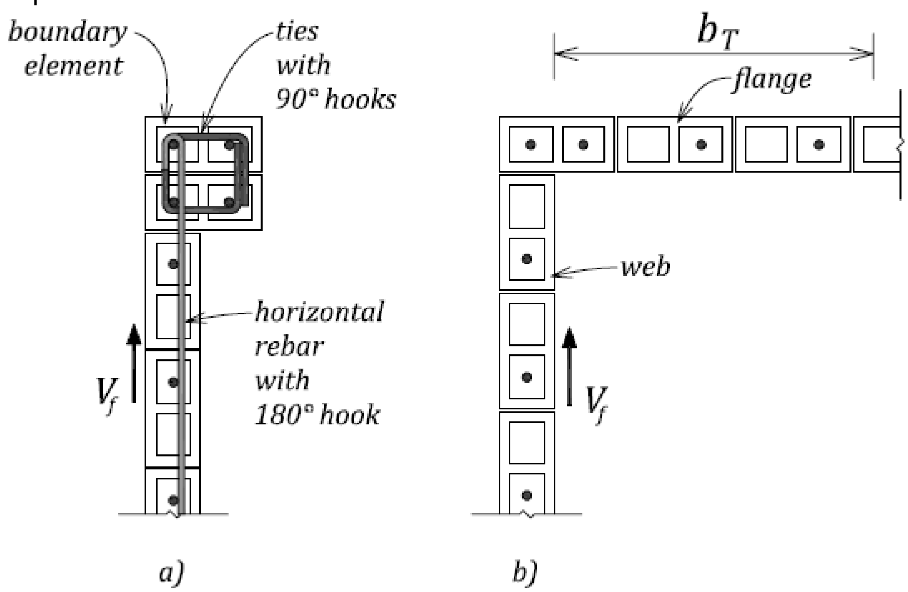


Figure 2-44. Boundary elements and flanges in reinforced masonry shear walls: a) boundary elements; b) flanged shear walls.

Paulay and Priestley (1992) proposed effective overhanging flange widths for reinforced concrete and reinforced masonry shear walls. For tension flanges, it is assumed that vertical forces due to shear stresses introduced by the web of the wall into the flange spread out at a slope of 1:2. For reinforced concrete flanged shear walls, the flexural strength of wall section with the flange in compression is insensitive to the effective flange width as the neutral axis is probably in the flange. After significant tension yield excursion in the flange, the compression contact area becomes rather small after load reversal, with outer bars toward the tips of the flange still in tensile strain.

As a result, the suggested the following overhanging flange width b_T to be used in seismic design for the flanges under tension and compression are as follows:

- Tension flange: $0.5h_w$
- Compression flange: $0.15h_w$

where h_w denotes the wall height. Note that these b_T values are different than the overhanging flange widths prescribed by CSA S304.1-04 for non-seismic design (see Table C-1 and Figure C-10 in Appendix C).

Shear walls with unsymmetrical flanges will have different flexural resistances, depending on whether flange acts in tension or in compression. Research studies on T-section walls have shown that such walls can exhibit larger ductility when the flanges are in compression. However, T- and L-section walls may have limited ductility when flanges are in tension (Paulay and Priestley, 1992; Priestley and Limin, 1995). Their experiments have shown that wall failure was sudden and brittle, and was initiated by a compression failure of the non-flange end of the wall, as shown in Figure 2-45 b). This was principally due to the large compression force needed to balance the large tension capacity of the reinforcement in the flange section.

In walls with unsymmetrical flanges, such as the T-section wall shown in Figure 2-45, the designer should be careful when applying the capacity design approach to determine flexural and shear capacity. The flexural capacity of the wall section is reached when the flange is in compression and the axial load is at minimum, $P_{f\,\min}$,as shown in Figure 2-45a. However, the maximum shear occurs when the flange is in tension and the axial load is at maximum, $P_{f\,\max}$, as shown in Figure 2-45b (this will result in a significantly higher flexural strength). A similar approach should be taken when the capacity design approach is applied to shear walls with pilasters.

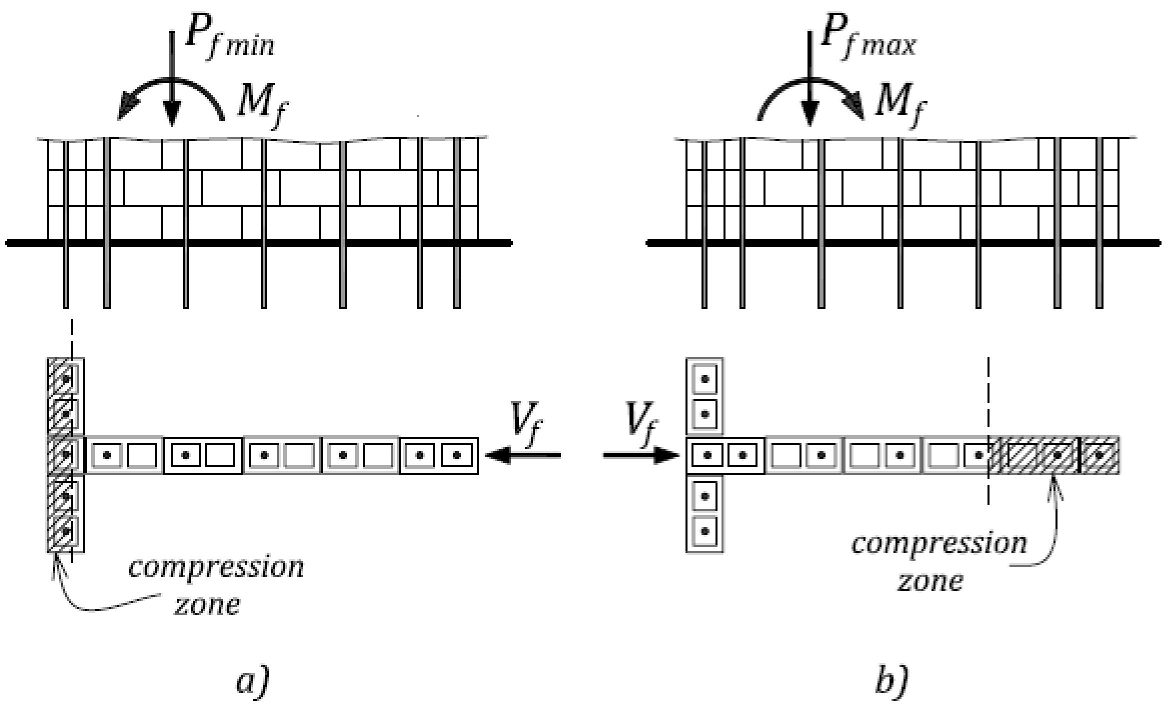


Figure 2-45. T-section flanged shear wall: a) flexural design scenario: web in tension; b) shear design scenario: web in compression.

The importance of web-to-flange connection for effective shear transfer in flanged shear walls is discussed in Section C.2. CSA S304.1-04 (clauses 7.11.1 to 7.11.3) prescribes three alternative approaches to achieve the effective shear transfer. Seismic studies in the U.S. under the TCCMAR research program resulted in recommendations related to horizontal reinforcement at the web-to-flange intersections (Wallace, Klingner, and Schuller, 1998). To ensure the effective shear transfer, horizontal reinforcement in bond beams needs to be continued from one wall into other, for a distance of 600 mm (2 feet) or 40 bar diameters, whichever is greater. The grout must be continued across the intersection by removing the face shells of the masonry units in one of the walls, as illustrated in Figure 2-46. Note that ACI 530-08 (Cl.1.17.3.2.6) requires that bond beams in ductile walls be provided at a vertical spacing of 1200 mm (4 feet).

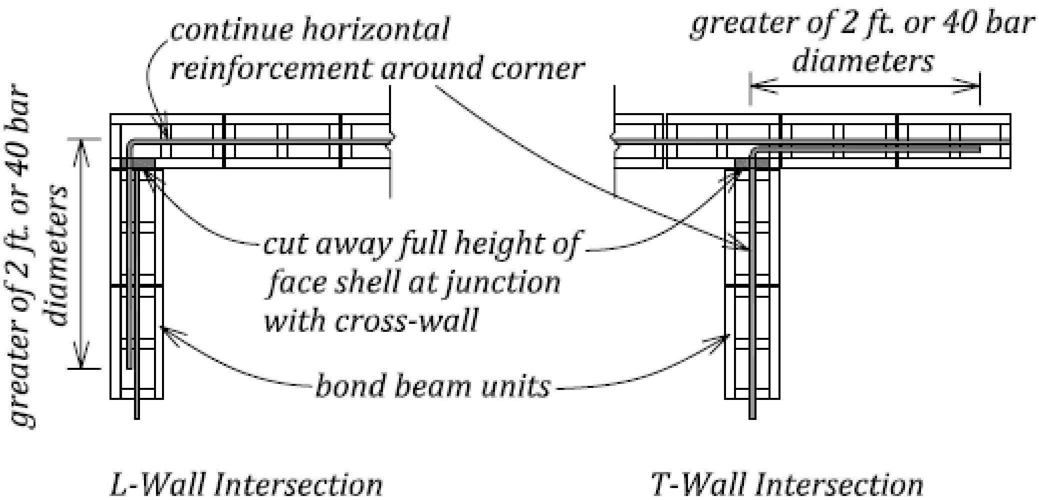


Figure 2-46. Horizontal reinforcement at the web-to-flange intersection: TCCMAR recommendations.

As an alternative to boundary elements, the New Zealand masonry standard NZS 4230:2004 Cl.7.4.6.5 prescribes the use of horizontal confining plates in ductile reinforced masonry walls. These thin perforated metal plates (made either of stainless steel or galvanized steel) are placed in mortar bed joints in the compression zone of rectangular walls. The confining plates are effective in increasing the maximum masonry compressive strain in plastic hinge regions to 0.008 (this value is significantly higher than the 0.0025 value prescribed by CSA S304.1-04 for unconfined walls). Provision of confining plates in the New Zealand masonry standard is based on the research done by Priestley (1982).

2.6.7 Wall-to-Diaphragm Anchorage

CSA A370-04

Masonry shear walls should be adequately anchored to floor and roof diaphragms.

The maximum anchor spacing between walls and horizontal lateral supports must not exceed ten times the nominal wall thickness (t+10 mm) (Cl.7.2.2). Anchors must be fully embedded in reinforced bond beams or reinforced vertical cells.

When the unfactored load applied normal to a wall is greater than 0.24 kPa, the ultimate strength of a wall anchor must not be less than 1,600 N (Cl.8.2.1).

Commentary

Anchorage is one of the most important, and in many cases the most vulnerable, components of existing masonry buildings exposed to earthquake effects. Many failures of masonry buildings result from wall-diaphragm failure that allows an out-of-plane wall failure, followed by a diaphragm failure.

Wall anchors must be effective in resisting the horizontal design forces from in-plane and out-of plane seismic loads. According to the capacity design approach, anchors should be designed to remain elastic in a seismic event (no yielding). This can be achieved by designing the anchor capacity based on the wall capacity, or on the elastic wall forces (corresponding to $R_d R_o$ of 1.0).

The anchors need to resist tension and shear forces, as shown in Figure 2-47.

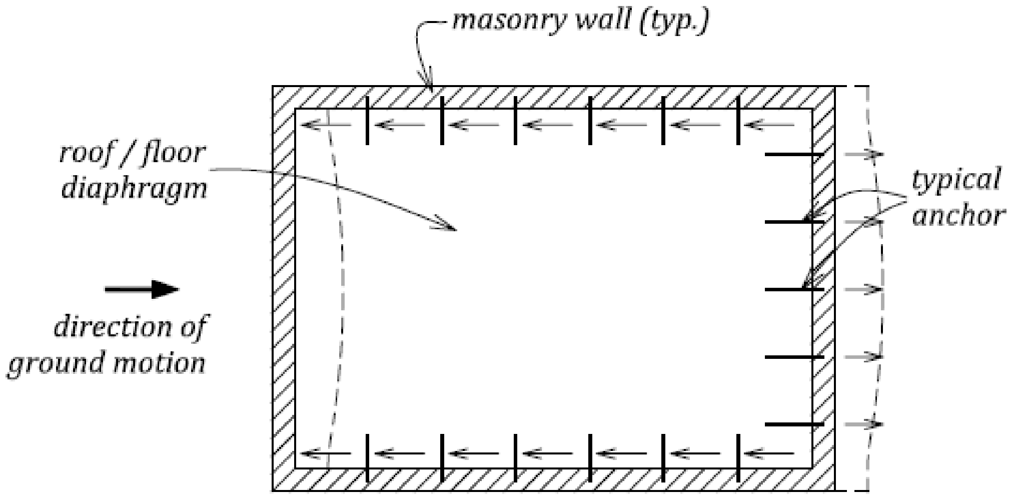


Figure 2-47. Tension and shear anchors at the wall-to-diaphragm connection.

Seismic load provisions for nonstructural components and their connections (including anchors) are provided in NBCC 2005 Cl.4.1.8.17.

2.6.8 Constructability Issues

Most of the information provided in this section has been adapted from the Masonry Technical Manual prepared by the Masonry Institute of BC (2008).

2.6.8.1 Reinforcement

Reinforced masonry is basically another form of reinforced concrete. However, reinforcing and grouting details should consider the core configuration of the masonry units. Care should be taken to disperse the rebar throughout the wall, and to avoid congestion in individual vertical cells. The cell size of the masonry units will dictate the size and number of bars that can be effectively grouted. A reinforcement arrangement, such as the one shown in Figure 2-48, is unsuitable and should be avoided. Typical reinforced masonry makes use of 15M or 20M bars. Units of 125, 150 and 200 mm nominal width should not contain more than one vertical bar per cell (2 bars at splices). 25M bars are occasionally used, but are more difficult to handle and require long laps. Vertical bars are typically placed in one layer in the centre of the wall. Site coordination is required to ensure that rebar dowels are installed to coincide with reinforced masonry cell locations.

Horizontal rebar is placed in bond beam courses using special bond beam blocks that have depressed or knock-out webs. Bond beams are typically spaced at 2400 mm vertically, but may also be positioned to coincide with lintel courses over openings. Joint reinforcement is often used in addition to bond beam bars. It is a ladder of 9 gauge (3.7 mm) galvanized wire installed in the mortar bed joint, which positions a wire in the centre of each block face shell. It is spaced at a maximum of 400 mm when used as seismic reinforcement. Joint reinforcement resists wall cracking and can contribute to the horizontal steel area in the wall. If joint reinforcement is not used, the maximum spacing of bond beams is 1200 mm for seismic detailing.

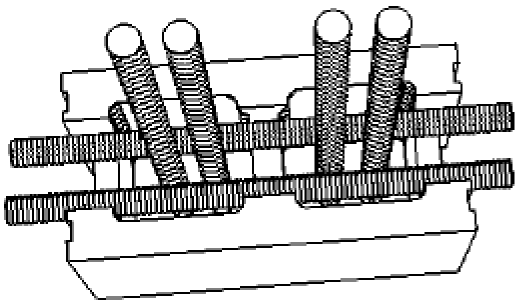


Figure 2-48. An example of inappropriate reinforcement arrangement: 2 bars vertically and 2 bars horizontally in a 20 cm wall are almost impossible to grout, particularly at splices where the steel is doubled (MIBC, 2008, reproduced by permission of the Masonry Institute of BC).

In addition to flexural, shear and minimum seismic steel, reinforcing is also required around openings over 1000 mm in loadbearing walls, at each side of control joints, and at the corners, ends, intersections and tops of walls. CSA S304.1-04 (Clause 4.6.1) allows unreinforced masonry partitions if they are less than 200 kg/m^2 in mass and 3 m in height, but only for seismic hazard indices $I_E F_a S_a(0.2) < 0.75$.

Nonloadbearing masonry partitions must have adequate top anchorage to avoid out-of-plane collapse. Dowels or angle clips must align with cells containing vertical bars (see CSA A370-04 for anchorage details). Bond beams at the tops of walls constructed under slabs or beams should be located in the second course below the slab to allow effective grouting of that course. Cells in the top course containing vertical bars can be dry packed with grout as they are laid with open-end units.

2.6.8.2 Masonry grout

Masonry grout, or “blockfill”, must flow for long distances through relatively small cells to anchor wall reinforcement. It is therefore placed at a much higher slump than regular concrete – in the range of 200 to 250 mm. While this water content would be problematic in cast-in-place concrete, in masonry the extra water necessary for placement is absorbed into the masonry units, thereby reducing the in-place water/cement ratio and providing adequate strength in the wall. Standard compressive strength tests using non-absorbent cylinders provide misleading data, as the extra water is trapped in the cylinder. Testing has shown the actual grout strength to be at least 50% higher than cylinder results. This situation is recognized in CSA S304.1 by basing masonry strength requirements on grout strengths of only 12.5 MPa by cylinder test. In some cases, a higher cement content grout (20 MPa) may be preferred for pumping reasons.

The most commonly used type of grout is Course Grout, which has a maximum aggregate size of 12 mm. Fine Grout uses coarse sand for aggregate and is usually used in small core units such as reinforced brick. Grout is supplied either by ready-mix truck or mixed on site, with quality control data available from the supplier or field test cylinders.

While grouting, care must be taken to completely fill the reinforced cores and to ensure that all bars, bolts and anchors are fully embedded. Vibration is usually not practical, but bars can be shaken to “puddle” the grout. Grout is often pumped in 2.4 m pours from bond beam to bond beam. The maximum pour height for “high-lift grouting” in CSA A371-04 Masonry Construction for Buildings is 4.5 m, but this should only be considered for H-block or 250 and 300 mm units. For total grout pours of 3 m or more, the grout must be placed in lifts of 2 m or less.

Sample base specification:

- Grout to meet CSA A179-04 requirements
- Minimum compressive strength 12.5 MPa at 28 days by cylinder test under the property specification
- Maximum aggregate size 12 mm diameter
- Grout slump 200 to 250 mm

2.6.8.3 Masonry mortar

Unlike reinforcing and grout, there are few issues in the specification, preparation and installation of mortar for structural masonry. CSA A179-04 covers mortar types and mixing, with Type S mortar almost always used for structural masonry. It provides the balance of mortar strength and bond that is required for good seismic performance. Unlike most cement-based products, compressive strength is not the dominant material criteria. Good bond results from mortar properties such as workability, adhesion, cohesion and water retention. Adequate bond binds the units together to provide structural integrity, tensile and shear capacity, and moisture resistance. In a mortar mix, Portland cement provides compressive strength and durability, while mortar cement, masonry cement or lime provides the properties that lead to good bond.

Most mortar is mixed on-site, and can be checked against the proportions specified in CSA A179-04. There are also pre-manufactured dry and wet mortars. The compressive cube strength required in CSA A179-04 for these products can be confirmed by plant or site test data. Site inspection of mortar mixing is generally not a significant concern for designers, because the bricklayer and the specifier are both looking for workable, well-proportioned mixes that provide installation efficiency for the mason, and long term performance for the designer. Mortar joints should be well filled and properly tooled for good performance. Concave tooled joints are the best shape for both structural and weather resistance.

Mortar joints compensate for minor dimensional variations in the masonry units, and provide coursing adjustment that may be necessary to meet required dimensions. Mortar joints also contribute to the architectural quality of the masonry assembly through colour and modularity.

2.6.8.4 Unit sizes and layout

Concrete masonry units are made in various sizes and shapes to fit different construction needs. Each size and shape is also available in various profiles and surface treatments. Concrete unit sizes are usually referred to by their nominal dimensions. Thus a unit known as 20 cm or 200x200x400 mm, will actually measure 190x190x390 mm to allow for 10 mm joints (see Figure 2-49). Standard nominal widths are 100, 150, 200, 250 and 300 mm, with 200 mm being the most common size for structural walls.

Working to a 200 mm module will minimize cutting, and maintain the alignment of vertical cells for rebar, as illustrated in Figure 2-50. Where possible, piers, walls and openings should be dimensioned in multiples of 200 mm. Foundation dowels must also be laid out to match the module of vertically reinforced cells.

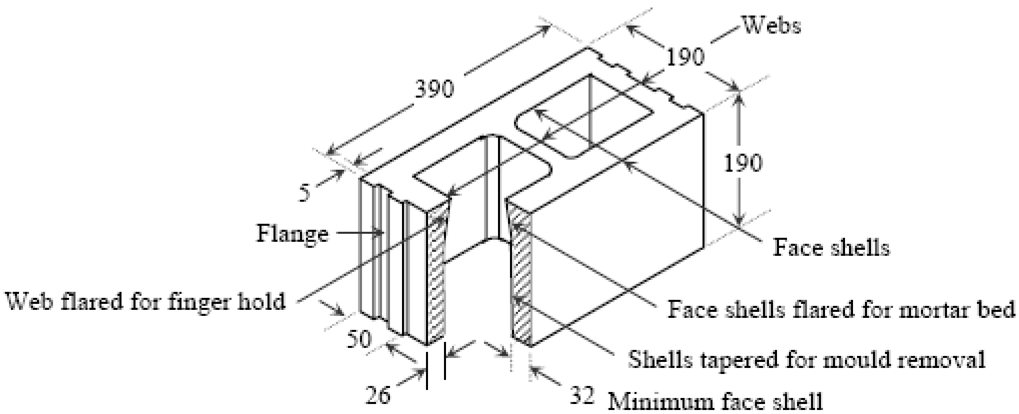


Figure 2-49. A typical 200 mm block unit (Hatzinikolas and Korany, 2005, reproduced by the authors' permission).

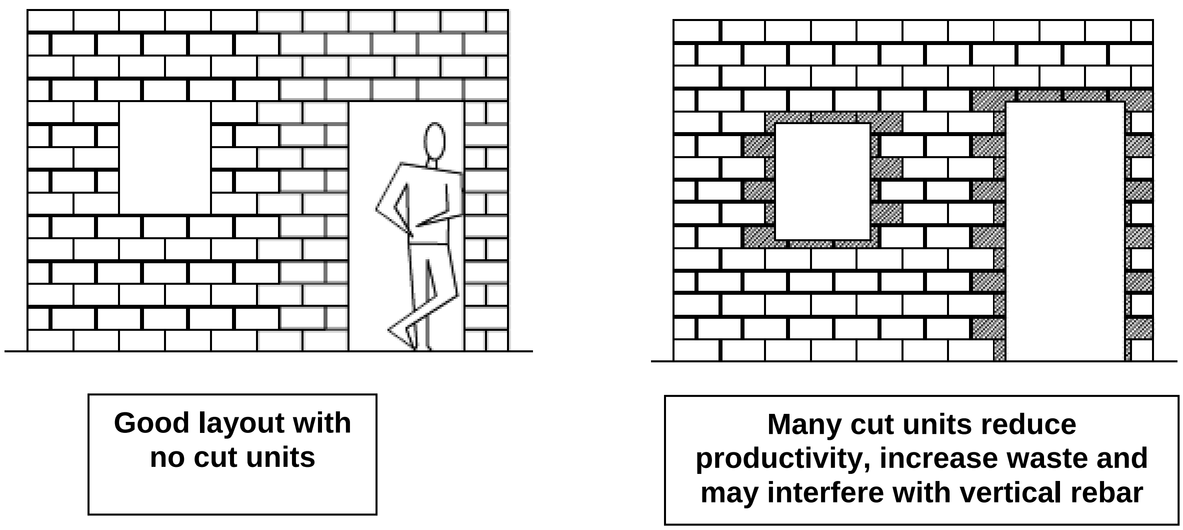


Figure 2-50. Examples of good and poor masonry layout (MIBC, 2008, reproduced by permission of the Masonry Institute of BC).

2.6.8.5 Other construction issues

In "high-lift grouting" (over 1.5 m), clean-out/inspection holes at the base of the reinforced cells may facilitate the removal of excessive mortar droppings and, more importantly, can confirm that grout has reached the bottom of the core. Clause 8.2.3.2.2 of CSA A371-04 allows the common practice of waiving the requirement for clean-out/inspection holes by the designer when the masonry contractor has demonstrated acceptable performance, or where the walls are not structurally critical. In some cases, the designer may require the initial walls to have clean-outs, pending demonstrated performance, and then waive them for the remaining walls.

Vertical movement joints in reinforced masonry walls are required to accommodate thermal and moisture movements, and possible foundation settlement. They are typically specified at a maximum spacing of 15 m.

Masonry walls should be installed to meet the requirements and tolerances of CSA A371-04 standard.

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3 Summary of Changes in NBCC 2005 and CSA S304.1-04 Seismic Design Requirements for Masonry Buildings

3.1 Introduction

This chapter summarizes the differences in seismic design provisions contained in the 1995 and 2005 editions of the NBCC, and the 1994 and the 2004 editions of CSA S304.1. Chapter 1 provides background on the seismic response of structures, seismic analysis methods, and the key NBCC 2005 seismic provisions of relevance for masonry design. Appendix A presents the NBCC 1995 seismic provisions and discusses changes in the two editions of the code. Chapter 2 provides an overview of the CSA S304.1 seismic design requirements for reinforced masonry walls.

This chapter also presents the results of a case study of a hypothetical warehouse building located in three Canadian cities characterized by different seismic risk (Vancouver, Calgary, and Toronto), based on both the NBCC 1995 and NBCC 2005.

3.2 Comparison of the Seismic Load Requirements of the 2005 and 1995 Editions of NBCC

NBCC 1995 and 2005 classified masonry shear walls based on their seismic performance requirements, as summarized in
Table 3-1.

Table 3-1. Classes of Reinforced Masonry Walls Based on Seismic Performance Requirements

NBCC 1995 Table 4.1.9.1.B and CSA S304.1-94	NBCC 2005 Table 4.1.8.9 and CSA S304.1-04	Comments
Unreinforced masonry R ***=1.0***	R_d =1.0 R_o =1.0	Slight difference in where unreinforced masonry could be used
Reinforced masonry R ***=1.5***	Shear walls with conventional construction R_d =1.5 R_o =1.5	No major changes in seismic design requirements in S304.1-04
Not defined	Limited ductility shear walls R_d =1.5 R_o =1.5	New class introduced in NBCC 2005 and S304.1-04
Reinforced masonry with nominal ductility R ***=2.0***	Moderately ductile shear walls R_d =2.0 R_o =1.5	No major changes in seismic design requirements in S304.1-04
Not defined	Moderately ductile squat shear walls R_d =2.0 R_o =1.5	New class introduced in NBCC 2005 and S304.1-04

Note that squat shear walls (height/length ratio less than unity) are common in low-rise masonry buildings, such as warehouses, schools and fire halls. Some of these buildings (e.g. fire halls), are defined as post-disaster facilities by NBCC 2005. A new restriction has been introduced in NBCC 2005 (Cl.4.1.8.10.2), that requires post-disaster facilities to have an SFRS with a R_d of

2.0 or higher. This provision means that squat masonry shear walls in post-disaster buildings must be designed to the CSA S304.1-04 provisions for "moderately ductile squat shear walls". A comparison of NBCC 1995 and NBCC 2005 seismic design provisions is presented in Table 3-2.

Table 3-2. Comparison of NBCC 1995 and NBCC 2005 Seismic Design Provisions - Equivalent Static Force Procedure

Provision	NBCC 1995	NBCC 2005
Analysis method	***Cl.4.1.9.1.(7c)*** Static method is the default method. Dynamic method may allow a decrease in the base shear.	***Cl.4.1.8.7*** Dynamic method is the default method; static method is restricted to certain structures and seismic hazard.
Seismic force	***Cl.4.1.9.1.(4,5)*** **V = vS(T)IFW / (R/U)**	***Cl.4.1.8.11*** $V = S(T) M_v I_e W / (R_d R_o)$
Base response spectrum	***Cl.4.1.9.1.(6)*** **v S** **v** – amplitude **S** – shape, dependent on ratio of Z_a/Z_v	***Cl.4.1.8.4*** $S(T)=F_a S_a(T)$ or $F_v S_a(T)$ $S_a(T)$ based on UHS
Site conditions	***Cl.4.1.9.1.(11)*** **F** Independent of **T** and **v**	***Cl.4.1.8.4*** F_a or F_v Depends on **T** and S_a
Importance of structure	***Cl.4.1.9.1.(10)*** **I**	***Cl.4.1.8.5*** I_E Same as NBCC1995
Inelastic response	***Cl.4.1.9.1.(8,9)*** **R/U** Implied overstrength	***Cl.4.1.8.9*** $R_d R_o$ Explicit overstrength
MDOF Forces from higher modes	***Cl.4.1.9.1.(6)*** Increase **S** in long period range **S** decreases slowly with **T** beyond 0.5 seconds	***Cl.4.1.8.11*** M_v multiplier on base shear Depends on period, type of structure and shape of $S_a(T)$
MDOF Distribution of forces	***Cl.4.1.9.1.(13)*** F_t Higher force in top storey	***Cl.4.1.8.11.(6)*** F_t Same as NBCC 1995
MDOF Overturning forces	***Cl.4.1.9.1.(23)*** **J** Moment reduction factor	***Cl.4.1.8.9.(7)*** **J** Revised for consistency with M_v
Eccentricity	***Cl.4.1.9.1.(28)*** $Tx=F_x(1.5e_x \pm 0.1D_{nx})$ or $Tx=F_x(0.5e_x \pm 0.1D_{nx})$	***Cl.4.1.8.11.(8,9,10)*** $Tx=F_x(e_x \pm 0.1D_{nx})$ Must determine torsional sensitivity
Irregularities	***Cl.4.1.9.3*** Mainly height restrictions and some specific restrictions on masonry	***Cl.4.1.8.6*** Irregularities better defined with more stringent requirements

3.3 Comparison of the Seismic Design Requirements of the 2004 and 1994 Editions of CSA S304.1

3.3.1 Summary of New Seismic Design Provisions in CSA S304.1-04

The classification of masonry walls has been expanded, and new definitions introduced in NBCC 2005 and CSA S304.1-04, as shown in Table 3.1. These changes provide similar definitions for masonry and concrete walls in the new standards.

NBCC 2005 imposes more height limitations than NBCC 1995. Walls with “limited ductility” are a new classification with the same R_d and R_o values as “conventional construction”. This classification allows design of limited ductility walls in taller buildings, however more stringent detailing is provided.

Moderately ductile squat shear walls are a new classification with an $R_d = 2.0$. They have less severe restrictions on height to thickness ratios, and require additional checks on horizontal reinforcement.

3.3.2 Comparison of the Seismic Design and Detailing Requirements for Reinforced Masonry Walls in CSA S304.1-04 and CSA S304.1-94

This section compares the seismic design and detailing requirements for classes of walls in the 1994 and 2004 editions of CSA S304.1 standard. The following classes of walls can be compared:

- “Moderately ductile shear walls” (S304.1-04) and “reinforced masonry with nominal ductility” (S304.1-94) – see Table 3-3, and
- “Shear walls with conventional construction” (S304.1-04) and “reinforced masonry” (S304.1-94).

The “limited ductility shear walls” and “moderately ductile squat shear walls”, did not exist in previous editions of CSA S304.1, so a comparison is not possible. For information on the seismic requirements for these wall classes see Table 2-4.

Table 3-3. Comparison of Seismic Design Requirements for Moderately Ductile Shear Walls (S304.1-04) and Reinforced Masonry with Nominal Ductility (S304.1-94)

Provision	CSA S304.1-94 Reinforced masonry with nominal ductility	CSA S304.1-04 Moderately ductile shear walls
Ductility level	R =2.0	R_d =2.0 R_o =1.5
Plastic hinge region	***Clause A5.2***	***Clause 10.16.5.2.1***
	l_p = greater of l_w or $h_w/6$	Unchanged
Ductility check	***Clause A.7***	***Clause 10.16.5.2.3***
	1. $\varepsilon_m = 0.0025$ *2.* $c/l_w < 0.2$ when $h_w/l_w < 3$	1. $\varepsilon_m = 0.0025$ 2. $c/l_w < 0.2$ when $h_w/l_w < 4$ $c/l_w < 0.15$ when $4 < h_w/l_w < 8$
Wall height-to-thickness ratio restrictions	***Clause A5.2***	***Clause 10.16.5.2.2***
	$h/(t+10) < 14$	Unchanged
Shear/diagonal tension resistance	***Clause A6.1***	***Clause 10.16.5.3.1***
	$V_r = 0.5V_m + V_s$ (50% reduction in the masonry shear resistance)	Unchanged
Sliding shear resistance	***Clause A6.2***	***Clause 10.16.5.3.2***
	$V_r = \phi_m \mu P_2$ only the reinforcement in the tension zone should be taken into account for P_2 calculation.	Unchanged
Grouting	***Clause A5.3***	***Clause 10.16.4.1.3***
	Masonry within the plastic hinge region shall be fully grouted.	Unchanged
Minimum seismic reinforcement requirements	**Clause 5.2.2**	***Clause 10.15.2.2***
	Minimum seismic reinforcement requirements apply	Unchanged

Note that shear walls with conventional construction (S304.1-04) and reinforced masonry walls (S304.1-94) do not require the special seismic detailing like limited ductility and moderate ductility walls. These walls need to be designed to resist the effect of factored loads, and to satisfy the minimum seismic reinforcement requirements summarized in Table 3-4. Under the NBCC 2005 Cl.4.1.8.1.1, seismic design requirements need to be considered when $S(0.2) \geq 0.12$. However, it is possible to use unreinforced masonry at sites where $I_E F_a S_a(0.2) < 0.35$ (S304.1-04 Cl.4.5.1).

Table 3-4. Comparison of CSA S304.1-94 and S304.1-04 Seismic Reinforcement Requirements for Shear Walls

	CSA S304.1-94	**CSA S304.1-04**
Applicability of minimum seismic reinforcement requirements	***Clause 6.3.3.1*** In velocity- or acceleration-related seismic zones of 2 and higher, reinforcement conforming to Clause 5.2.2 shall be provided for masonry construction in loadbearing and lateral load-resisting masonry	***Clause 4.6.1*** At sites where the seismic hazard index $I_E F_a S_a(0.2) \geq 0.35$, reinforcement conforming to Clause 10.15.2 shall be provided for masonry construction in loadbearing and lateral load-resisting masonry
Minimum area: vertical & horizontal Reinforcement	***Clause 5.2.2.2*** Loadbearing walls and shear walls shall be reinforced horizontally and vertically with steel having a minimum total area of $0.002A_g$ distributed as follows: $A_v = 0.002A_g\alpha$ $A_h = 0.002A_g(1-\alpha)$ Where A_v = area of vertical steel A_h = area of horizontal steel α = distribution factor between 0.33 and 0.67, at the discretion of the designer.	***Clause 10.15.2.2*** (Same requirements in different terms) Loadbearing walls (including shear walls) shall be reinforced horizontally and vertically with steel having a minimum total area of $A_{stotal} = 0.002A_g$ distributed with a minimum area in one direction of at least $A_{v\min} = 0.00067A_g$ (approximately one-third of the total area)

	CSA S304.1-94	CSA S304.1-04
Spacing: vertical reinforcement	***Clause 5.2.2.2*** Vertical reinforcement shall be spaced at not more than a) 6 times the wall thickness or b) 1200 mm	***Clause 10.16.4.3.2*** Vertical seismic reinforcement shall be uniformly distributed over the length of the wall. Its spacing shall not exceed the lesser of a) $6(t+10)$ mm b) 1200 mm c) $l_w/4$ (for limited ductility or moderately ductile walls only) but it need not be less than 600 mm
Spacing: horizontal reinforcement	***Clause 5.2.2.2*** Horizontal reinforcement shall be spaced at not more than c) 6 times the wall thickness or d) 1200 mm When joint reinforcement is provided, the spacing should not exceed 400 mm (this is not clearly specified by S304.1).	Outside plastic hinge regions (Cl.10.15.2.6): Horizontal seismic reinforcement shall be continuous between lateral supports. Its spacing shall not exceed a) 400 mm where only joint reinforcement is used; b) 1200 mm where only bond beams are used; or c) 2400 mm for bond beams and 400 mm for joint reinforcement where both are used. Plastic hinge regions (Cl. 10.16.4.3.3): Reinforcing bars are to be used in the *plastic hinge region*, at a spacing not more than a) 1200 mm or b) $l_w/2$

3.4 Comparison of Masonry Wall Design for Different Design Codes and Site Locations

3.4.1 Building Description

Two typical shear walls, one squat and one flexural (non-squat), are considered for a single-storey reinforced masonry warehouse. The reinforcement required by NBCC 2005 and CSA 304.1-04 is compared to the reinforcement required by NBCC 1995 and CSA 304.1-94. The example warehouse is 64 m long and 27 m wide, with a wall height of 6.6 m. Masonry walls are located around the perimeter, with steel columns in the interior. The roof structure consists of steel beams, open web steel joists, and a composite steel and concrete deck. The design is presented for: Vancouver, BC; Calgary, AB; and Toronto, ON.

3.4.2 Design Criteria

1. Lateral seismic forces are calculated using the NBCC 2005 and NBCC 1995 (wind loads were not considered)
2. Masonry walls are designed to CSA S304.1-94 and CSA S304.1-04 for in-plane seismic loads (slenderness effects not checked in the design)
3. Masonry properties: 190 mm hollow concrete block units, block strength 15 MPa, and Type S mortar
4. Reinforcement properties: Grade 400 steel for vertical reinforcement and horizontal bond beam reinforcement, and ladder-type wire (No.9 ASWG) joint reinforcement

3.4.3 NBCC Seismic Load Calculations

The seismic weight (W) is calculated as 7370 kN, and includes the dead load and 25% of the snow load. For consistency, the same seismic weight has been taken for all locations, despite the difference in actual design snow loads. The upper half of the walls is included in the seismic weight calculation, and they are assumed to be fully grouted (conservative assumption for the weight calculation only). The fundamental period has different values depending on the code: NBCC 2005 gives a period of 0.2 sec, while NBCC 1995 gives 0.07 sec and 0.11 sec for the main directions.

The roof diaphragm is considered to be rigid in the design. The building is symmetrical in plan with regard to both principal axes. The effects of accidental torsion are taken into account by increasing the in-plane seismic force along the sides of the building by 10%.

NBCC 1995 and NBCC 2005 seismic design parameters used for this study are summarized in tables below.

Table 3-5. An Overview of the NBCC 1995 and NBCC 2005 Design Parameters

Code	**NBCC 1995**	**NBCC 2005**
Ductility Level	Reinforced masonry R = 1.5	Shear walls with conventional construction R_d = 1.5 R_o = 1.5
Soil conditions	F= 1.5	Site Class D
Building Importance	Normal importance- all other buildings (Cl.4.1.9.1.10) I = 1.0	Normal importance (Table 4.1.2.1) I_E =1.0

Table 3-6. NBCC 2005 Seismic Design Parameters (Site Class D)

Location	$S_a(0.2)$ (Table C-2, Appendix C)	F_a (Table 4.1.8.4B)	**Seismic hazard index** $I_E F_a S_a(0.2)$
Vancouver	0.96	1.1	1.06 >0.35
Toronto	0.28	1.3	0.36>0.35
Calgary	0.15	1.3	0.20<0.35

Table 3-7. NBCC 1995 Seismic Design Parameters (Foundation factor F=1.5)

Location	Z_a	Z_v	Z_a/Z_v	S	$S \cdot F$	v
Vancouver	4	4	1	3	3	0.2
Toronto	1	0	1	3	3	0.05
Calgary	0	1	<1	2.1	3	0.05

3.4.4 Shear Wall Design

The dimensions of the typical squat and flexural shear walls are shown in Figure 3-1. The dimensions and material properties are the same for all three locations. The axial loads are slightly different (150 kN for the flexural wall and 230 kN for the squat wall). Note that the height/length aspect ratios are equal to 0.83 and 2.20 for the squat and flexural walls respectively.

The following material properties were used in the design:

- 200 mm nominal width concrete masonry units (190 mm actual)
- Masonry compressive strength: f'_m = 9.8 MPa for hollow ungrouted masonry, and f'_m =7.5 MPa for fully grouted masonry (15 MPa block)
- Steel yield strength: f_y = 400 MPa (used both for Grade 400 steel bars and joint reinforcement)

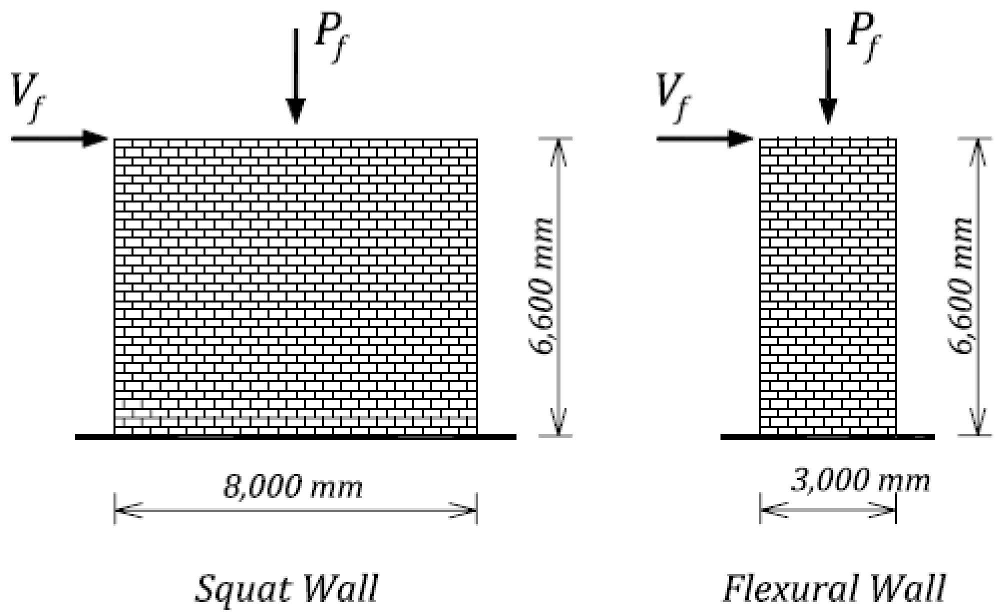

Figure 3-1. Wall dimensions and loading scheme.

The wall design parameters and key results are summarized in the following tables. Note that the vertical reinforcement is specified in terms of the number of bars of a specific size; this is different from a typical design specification, where the same information would be presented in terms of bar size and spacing.

Table 3-8. Design Results – Squat Shear Wall

Location	Shear Force V_f (kN)		Vertical and Horizontal Reinforcement	
	NBCC 1995	**NBCC 2005**	**NBCC 1995 S304.1-94**	**NBCC 2005 S304.1-04**
Vancouver	531	630	V:14-15M (*) ρ_v =0.18% H:15M@600 ρ_h =0.18%	V:16-15M (*) ρ_v =0.21% H:15M@600 ρ_h =0.18%
Toronto	133	185	V:4-15M (***) ρ_v =0.05% H: none ρ_h =0	V:8-15M (*,**) ρ_v =0.11% H:15M@2400+ ρ_h =0.10% 9 ga. joint reinf @200
Calgary	133	100	V:4-15M (***) ρ_v =0.05% H: none ρ_h =0	V:4-15M (***) ρ_v =0.05% H: none ρ_h =0

Notes:

V – vertical reinforcement ρ_v = vertical reinforcement ratio

H – horizontal reinforcement ρ_h = horizontal reinforcement ratio

*-fully grouted wall based on shear design

** - minimum seismic reinforcement requirements govern, corresponding to ρ_{total} =0.2% (S304.1-04 Cl.10.15.2.2)

***-minimum reinforcement requirements for loadbearing walls govern (S304.1-04 Cl.10.15.1.2)

Table 3-9. Design Results – Flexural Shear Wall

Location	Shear Force V_f (kN)		Vertical and Horizontal Reinforcement	
	NBCC 1995	NBCC 2005	NBCC 1995 S304.1-94	NBCC 2005 S304.1-04
Vancouver	152	180	V: 13-15M (*,**) ρ_v =0.5% H:15M@1200 ρ_h =0.09%	V: 15-15M (*,**) ρ_v =0.5% H: 15M@1200 ρ_h =0.09%
Toronto	38	55	V: 2-15M ρ_v =0.07% H: none ρ_h =0	V: 4-15M ρ_v =0.14% H: 15M@1200 ρ_h =0.09%
Calgary	38	30	V: 2-15M ρ_v =0.07% H: none ρ_h =0	V: 2-15M ρ_v =0.07% H: none ρ_h =0

Notes: V – vertical reinforcement H – horizontal reinforcement

ρ_v = vertical reinforcement ratio ρ_h = horizontal reinforcement ratio

*-fully grouted wall based on shear design

** - minimum seismic reinforcement requirements govern, corresponding to ρ_{total} =0.2% (S304.1-04 Cl.10.15.2.2)

3.4.5 Discussion

3.4.5.1 Design to NBCC 2005 and CSA S304.1-04

CSA S304.1-04 Cl.4.6.1 requires that minimum seismic reinforcement be provided when the seismic hazard index $I_E F_a S_a(0.2) \geq 0.35$ (Cl.10.15.2.2) (see Table 3-4). This applies to the wall designs for Vancouver and Toronto, but not Calgary. However, since these are loadbearing walls, the Calgary design must meet the minimum reinforcement requirements for loadbearing walls (Cl.10.15.1.1). Reinforcement requirements for the walls at the three locations are summarized in Table 3-10.

Table 3-10. CSA S304.1-4 Requirements for Shear Wall Reinforcement

Location	Unreinforced masonry Cl.4.6.1	Minimum reinf. required if walls are loadbearing Cl.10.15.1	Minimum seismic reinf. requirements Cl.10.15.2	Beyond minimum seismic reinf. requirements
Vancouver	Not possible	No, must meet seismic reinforcement requirements	Yes	Depends on the specific design
Toronto	Possible in some locations depending on site soil class	Yes, if reinforcement is required by design	Yes, if $I_E F_a S_a(0.2) \geq 0.35$	Depends on the specific design
Calgary	Possible for most locations	Yes, if reinforcement is required by design	Yes, if $I_E F_a S_a(0.2) \geq 0.35$	Depends on the specific design

3.4.5.2 Design to NBCC 1995 and CSA S304.1-94

CSA S304.1-94 Cl.6.3.3.1 required that minimum seismic reinforcement be provided for velocity- or acceleration-related seismic zones of 2 and higher (Cl.5.2.2) (see Table 3-4). This applies to the Vancouver design, but not to the Calgary or Toronto designs. However, since these shear walls are also loadbearing walls, the Toronto and Calgary designs must meet the minimum reinforcement requirements for loadbearing walls (Cl.5.2.1). It should be noted that S304.1-94 permitted the use of unreinforced masonry for Calgary and Toronto designs, provided that the tensile and compressive stresses were less than the permitted values.

3.4.5.3 Key Differences in the Designs

Squat wall (Table 3-8):

- Minor difference for Vancouver vertical reinforcement (16-15M bars for the NBCC 2005 design versus 14-15M bars for the NBCC 1995 design)
- An increase in vertical reinforcement for Toronto (8-15M bars for NBCC 2005 design versus 4-15M bars for NBCC 1995 design), plus the need to provide horizontal reinforcement to meet minimum S304.1-04 seismic reinforcement requirements (15M@2400 mm bond beam reinforcement and joint reinforcement at 200 mm spacing)
- No difference for Calgary

Flexural (non-slender) shear wall (Table 3-9):

- No difference for Vancouver
- An increase in vertical reinforcement in Toronto (4-15M bars for the NBCC 2005 design versus 2-15M bars for the NBCC 1995 design), plus the need to provide horizontal reinforcement to meet minimum S304.1-04 seismic reinforcement requirements (15M@1200 mm bond beam reinforcement versus 2-15M@2400 mm (note that S304.1-04 limits horizontal reinforcement spacing to maximum 1200 mm in the plastic hinge region)
- No difference for Calgary

3.4.5.4 Influence of Site Class and Building Importance

CSA S304.1-04 minimum seismic reinforcement requirements must be satisfied at locations where the seismic hazard index $I_E F_a S_a(0.2) \geq 0.35$ (Cl.4.6.1). The provision of minimum seismic reinforcement at a particular location is governed by the site class and the building importance (expressed through seismic importance factor I_E). Site classes B to E are considered as the most relevant for design purposes. Note that the fundamental period (T) is taken equal to 0.2 sec, which is typical for low-rise masonry buildings. The results for the three locations are summarized in Tables 3-11 to 3-13.

Note that the shaded cells indicate designs for which the S304.1-04 minimum seismic reinforcement requirements apply.

The following observations relate to the seismic hazard index values and the resulting seismic reinforcement requirements for parameters considered in this study:

- Vancouver site requires minimum S304.1-04 seismic reinforcement for all site classes and building importance levels
- Toronto site chosen for this study requires minimum S304.1-04 seismic reinforcement for many cases; note that the Toronto site chosen for this study has higher seismicity ($S_a(0.2)$ of 0.28) compared to some other sites in the Metro Toronto region (see Table 3-6), and that the results might be different for sites characterized by lower seismicity (more similar to Calgary)
- Calgary site does not require minimum seismic reinforcement for most cases (except for the site class E for higher importance buildings)

*Table 3-11. **Vancouver:** Seismic Hazard Index $I_E F_a S_a(0.2)$ for Different Site Classes and Building Importance Factors ($S_a(0.2)$ = 0.96)*

Site Class	F_a	Seismic Hazard Index		
		I_E=1.0	I_E=1.3	I_E=1.5
B	1.0	0.96	1.25	1.44
C	1.0	0.96	1.25	1.44
D	1.1	1.06	1.38	1.59
E	0.9	0.86	1.12	1.29

*Table 3-12. **Toronto:** Seismic Hazard Index $I_E F_a S_a(0.2)$ for Different Site Classes and Building Importance Factors ($S_a(0.2)$ = 0.28)*

Site Class	F_a	Seismic Hazard Index		
		I_E=1.0	I_E=1.3	I_E=1.5
B	0.8	0.22	0.29	0.34< 0.35
C	1.0	0.28	0.36	0.42
D	1.3	0.36	0.47	0.54
E	2.0	0.56	0.73	0.84

*Table 3-13. **Calgary:** Seismic Hazard Index $I_E F_a S_a(0.2)$ for Different Site Classes and Building Importance Factors ($S_a(0.2)$ = 0.15)*

Site Class	F_a	Seismic Hazard Index		
		I_E=1.0	I_E=1.3	I_E=1.5
B	0.8	0.12	0.16	0.18
C	1.0	0.15	0.20	0.23
D	1.3	0.20	0.26	0.30
E	2.1	0.32<0.35	0.42	0.48

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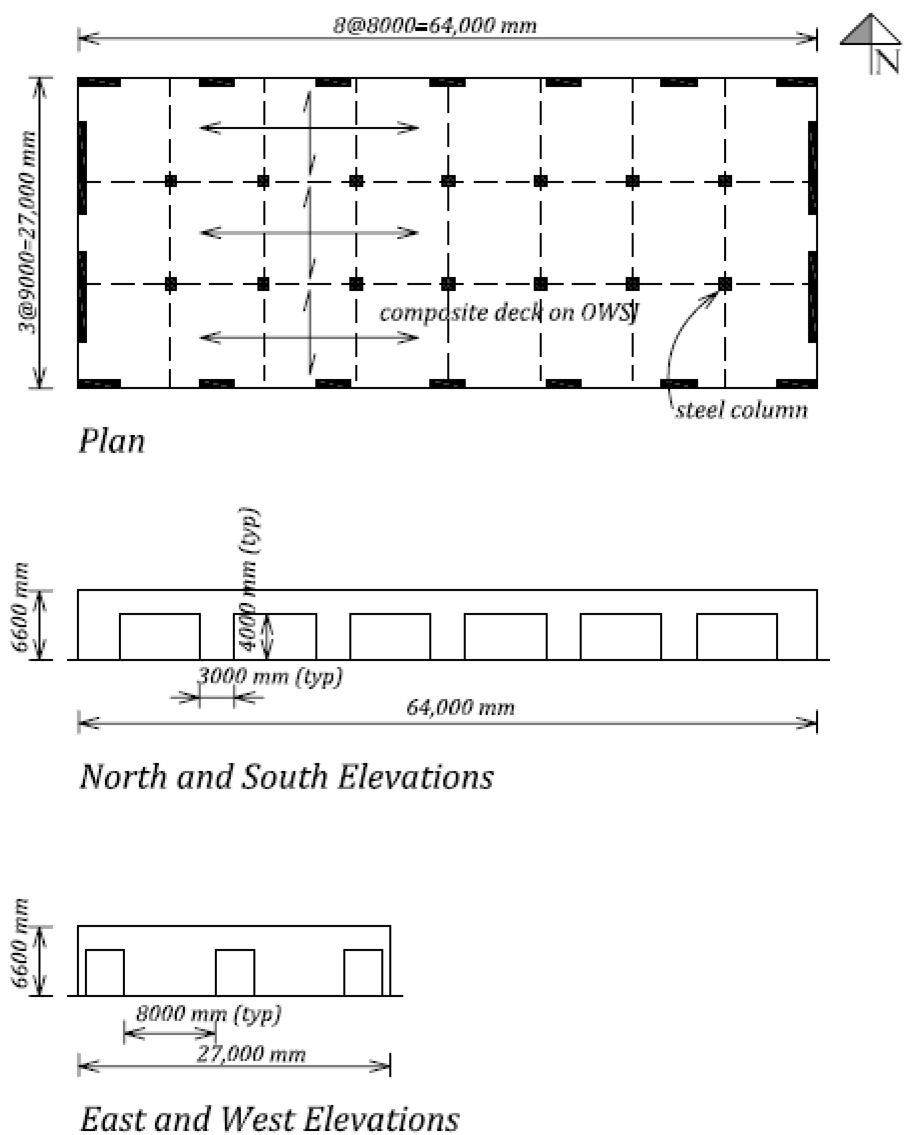
4 Design Examples

<u>EXAMPLE 1</u>: Seismic load calculation for a low-rise masonry building to NBCC 2005

Consider a single-storey warehouse building located in Mississauga, Ontario. The building plan dimensions are 64 m length by 27 m width, as shown on the figure below. The roof structure consists of steel beams, open web steel joists, and a composite steel and concrete deck with 70 mm concrete topping. The roof is supported by 190 mm reinforced block masonry walls at the perimeter and interior steel columns. The roof elevation is 6.6 m above the foundation. The soil at the building site is classed as a Site Class D per NBCC 2005.

Calculate the seismic base shear force for this building to NBCC 2005 seismic requirements (considering the masonry walls to be detailed as "conventional construction"). Next, determine the seismic shear forces in the walls, including the effect of accidental torsional eccentricity. Assume that the roof acts like a rigid diaphragm.



SOLUTION:

1. Calculate the seismic weight W (NBCC 2005 Cl.4.1.8.2)

a) Roof loads:

- Snow load (Mississauga, ON) W_s = 0.25*(1.1*0.8+0.4)= 0.32 kPa

(25% of the total snow load is used for the seismic weight)

- Roof self-weight (including beams, trusses, steel deck, roofing, insulation, and 65 mm concrete topping) W_D = 2.60 kPa

Total roof seismic weight W_{roof} = (0.32kPa+2.60kPa)(64.0m*27.0m)= 5046 kN

b) Wall weight:

Assume solid grouted walls w = 4.0 kN/m^2

(this is a conservative assumption and could be changed later if it is determined that partially grouted walls would be adequate)

The usual assumption is that the weight of all the walls above wall midheight is part of the seismic weight (mass) that responds to the ground motion and contributes to the total base shear.

Tributary wall surface area:

- North face elevation = 0.5*7*3.0m*6.6m + (64m-7*3m)*(6.6m-4.0m)= 181.1 m^2
- South face elevation (same as north face elevation) = 181.1 m^2
- East face elevation = 0.5*2*8.0m*6.6m + (27m-2*8m)*(6.6m-4.0m) = 81.4 m^2
- West face elevation (same as east face elevation) = 81.4 m^2

Total tributary wall area $Area$ = 525.0 m^2

Total wall seismic weight $W_{wall} = w * Area$ = 4.0*525.0= 2100 kN

The total seismic weight is equal to the sum of roof weight and the wall weight, that is,

$$W = W_{roof} + W_{wall} = 5046+2100 = 7146 \text{ kN} \approx 7150 \text{ kN}$$

2. Determine the seismic hazard for the site (see Section 1.5.2).

- Location: Mississauga, ON

  $S_a(0.2)$ = 0.31 (NBCC 2005 Appendix C, page C-21)

  $S_a(0.5)$ = 0.15

  $S_a(1.0)$ = 0.055

  $S_a(2.0)$ = 0.017

- Foundation factors

  F_a = 1.28 for $S_a(0.2)$ =0.31 and Site Class D (by interpolation from Table 1-10 or NBCC 2005 Table 4.1.8.4.B, since F_a = 1.3 for $S_a(0.2) \leq$ 0.25 and F_a = 1.2 for $S_a(0.2) =$ 0.50

  F_v = 1.4 for $S_a(1.0)$ = 0.055 and Site Class D (from Table 1-11 or NBCC 2005 Table 4.1.8.4.C), since F_v = 1.4 for $S_a(1.0) \leq$ 0.1

- Site design spectrum $S(T)$ (see Section 1.5.2)

For T =0.2 sec: $S(0.2) = F_a S_a(0.2)$= 1.28*0.31=0.40 $\boxed{S(0.2)=0.40}$

For T =0.5 sec: use the smaller of;

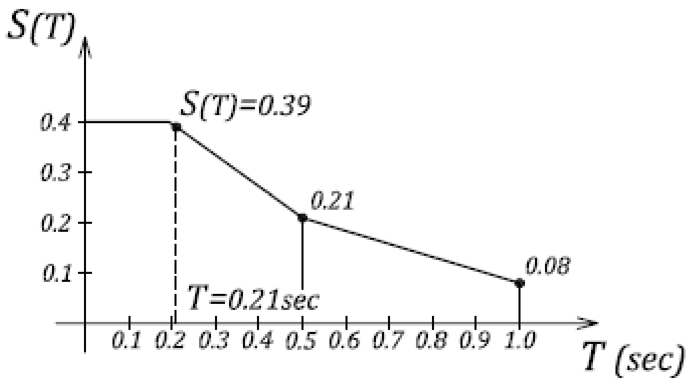
$$S(0.5) = F_v S_a(0.5) = 1.4*0.15 = 0.21 \quad \text{or}$$

$S(0.5) = F_a S_a(0.2)$=1.28*0.31=0.40, thus $\boxed{S(0.5)=0.21}$

For T =1 sec: $S(1.0) = F_v S_a(1.0)$=1.4*0.055=0.08 $\boxed{S(1.0)=0.08}$

For T =2 sec: $S(2.0) = F_v S_a(2.0)$=1.4*0.017=0.024 $\boxed{S(2.0)=0.024}$

The site design spectrum $S(T)$ is shown below.



- Building period (T) calculation (see Section 1.5.4 and NBCC 2005 Cl.4.1.8.11.(3).c) for wall structures)

  h_n = 6.6 m building height

  $T = 0.05(h_n)^{3/4}$ = 0.21 sec

Then interpolate between $S(0.2)$ and $S(0.5)$ to determine the design spectral acceleration:

$S(T)$=$S(0.21)$= 0.39

3. Compute the seismic base shear (see Section 1.5.4)

The base shear is given by the expression (NBCC 2005 Cl.4.1.8.11)

$$V = \frac{S(T) M_v I_E}{R_d R_o} W$$

where

I_E = 1.0 (building importance factor, equal to 1.0 for normal importance, 1.3 for high importance, and 1.5 for post-disaster buildings)

M_v = 1.0 (higher mode factor, equal to 1.0 for $T \leq 1.0$ sec, that is, most low-rise masonry buildings)

Building SFRS description: masonry structure – conventional construction (see Table 1-13 or NBCC 2005 Table 4.1.8.9), hence R_d = 1.5 and R_o = 1.5

The design base shear V is given by:

$$V = \frac{S(T) M_v I_E}{R_d R_o} W = \frac{0.39 * 1.0 * 1.0}{1.5 * 1.5} W = 0.17W$$

but not less than

$$V_{\min} = \frac{S(2.0) M_v I_E W}{R_d R_o} = \frac{0.024 * 1.0 * 1.0}{1.5 * 1.5} W = 0.0011W$$

and need not be taken more than

$$V_{max} = \left(\frac{2S(0.2)}{3}\right)\left(\frac{I_E W}{R_d R_o}\right) = \left(\frac{2*0.40}{3}\right)\left(\frac{1.0}{1.5*1.5}\right)W = 0.12W$$, provided $R_d \geq 1.5$.

The upper limit on the design seismic base shear governs and therefore
$V = 0.12W = 0.12*7150 = 858 \approx 860$ kN

Note that the upper limit on the base shear is often going to govern for low-rise masonry structures which have low fundamental periods. The lower bound value would generally only apply to very tall buildings.

4. Determine if the equivalent static procedure can be used (see Section 1.5.3 and NBCC 2005 Cl. 4.1.8.7).

According to the NBCC 2005, the dynamic method is the default method of determining member forces and deflections, but the equivalent static method can be used if the structure meets <u>any</u> of the following criteria:

(a) is located in a region of low seismic activity where the seismic hazard index $I_E F_a S_a(0.2) < 0.35$.

In this case, the seismic hazard index is $I_E F_a S_a(0.2)$=1.0*1.28*0.31=0.40 > 0.35 and so this criterion is not satisfied.

(b) is a regular structure less than 60 m in height with period T < 2 seconds in either direction.

This building is clearly less than 60 m in height and the period T < 2 sec (as discussed above). A structure is considered to be regular if it has none of the irregularities discussed in Table 1-15 of Section 1.5.10.1. A single storey structure by definition will not have any irregularities of Type 1 to 6. It does not have a Type 8 irregularity (non-orthogonal system) but could have a Type 7 irregularity (torsional sensitivity), and so this criterion may or may not be satisfied, depending on the torsional sensitivity.

(c) has any type of irregularity, other than Type 7, and is less than 20 m in height with period T < 0.5 seconds in either direction.

This structure satisfies the height and period criteria.

Since the criterion c) has been satisfied, the design can proceed by using the equivalent static analysis procedure. It will be shown later that, even when using a conservative assumption, the torsional sensitivity parameter B=1.2<1.7. Thus criterion b) would also be satisfied. For structures with the lateral resisting elements distributed around the perimeter walls the B value will almost always be less than 1.7.

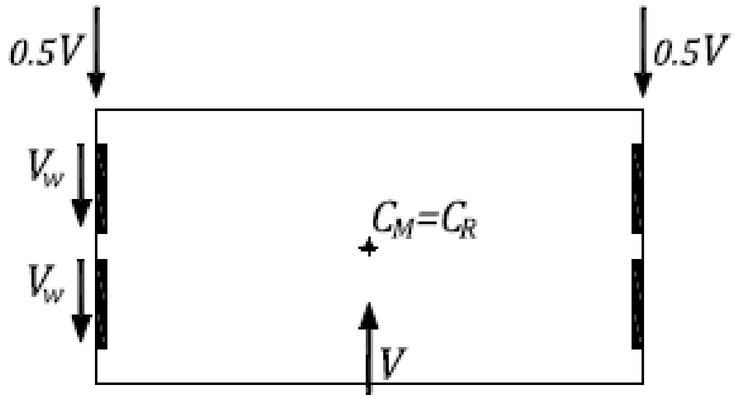
5. Distribute the base shear force to the individual walls.

In this example, the structure is symmetric in each direction and so the centre of mass, C_M, and the centre of resistance, C_R, coincide at the geometric centre of the structure. One might argue that in this simple system with walls at only each side of the building, the system is statically determinate in each direction and the total shear on each side can be determined using statics. However, how much shear goes to each of the walls on a side depends on the relative stiffness of the walls, although once yielding occurs the force on each wall depends on the yield strength of the wall.

a) Seismic forces in the N-S direction - no torsional effects (seismic force is assumed to act through the centre of resistance)

Since it is assumed that the roof diaphragm is rigid, the forces are distributed to the walls in proportion to wall stiffness. All walls in the N-S direction have the same geometry (height, length, thickness) and mechanical properties and it can be concluded that these walls have the same stiffness.

As a result, equal shear force will be developed at each side. The force per side is equal to (see the figure):
$0.5V = 0.5 * 860 = 430$ kN
So, shear force in each of the two walls in the N-S direction is equal to:
$V_V = \frac{0.5V}{2} = \frac{430}{2} = 215$ kN



b) Seismic forces in the N-S direction taking into account the effect of accidental torsion
The building is symmetrical in plan and so the centre of mass C_M coincides with the centre of resistance C_R (see Section 1.5.9 for more details on torsional effects). Therefore, there are no actual torsional effects in this building. However, NBCC 2005 Cl.4.1.8.11.(8) requires that torsional moments (torques) due to accidental eccentricities must be taken into account in the design. The forces due to accidental torsion can be determined by applying the seismic force at a point offset from the C_R by an accidental eccentricity $e_a = 0.1D_{nx}$, thereby causing the torsional moments equal to
$T_x = \pm V(0.1D_{nx}) = \pm 860 * (0.1 * 64.0) = \pm 5504$ kNm
Note that $D_{nx} = 64.0$ m (equal to the total length of the structure in the East/West direction).

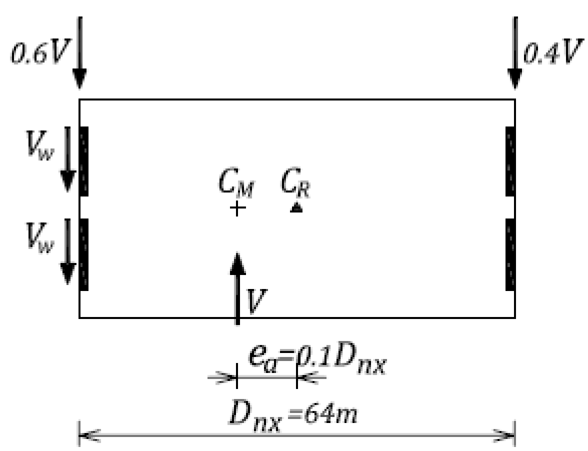
As a result of the accidental torsion, seismic shear forces resisted by each side of the building are different. These forces can be calculated by taking the sum of moments around the C_R (torsional moment created by force must be equal to the sum of moments created by the side forces). The resulting end forces are equal to $0.6V$ and $0.4V$, thereby indicating an increase in the end forces by $0.1V$ due to accidental torsion.

It should be noted that, in this example, accidental torsion would cause forces in the E-W walls as well because of the rigid diaphragm. But a conservative approach is to ignore the contribution of E-W walls and take all the torsional forces on the N-S walls.

The shear force in each N-S wall from accidental torsion is equal to:
$V_T = \frac{T/D_{nx}}{2} = \frac{5504/64}{2} = 43$ kN

Thus the maximum shear force in each of the two walls is the sum of the lateral component plus the torsional force,
$V_W = V_V + V_T = 215 + 43 = 258$ kN

Note that the same result could be obtained by applying the lateral load through a point equal to the accidental eccentricity to one side of the centre of rigidity and then solving for the wall forces using statics (see the figure). This would show that
$V_W = \frac{V}{2} * 0.6 = \frac{860}{2} * 0.6 = 258$ kN



Therefore, even though this building is symmetrical in plan, the accidental torsion causes increased seismic shear force in each wall of 43 kN, corresponding to a 20% increase compared to the design without torsion. However, this is based on the assumption that the N-S walls resist all the torsion. Walls in the E-W direction would also resist the torsional forces, and in this example the contribution to total torsional stiffness would be roughly the same for the E-W and N-S walls. Thus one could reduce the torsional forces on the N-S walls by roughly one half.

c) Seismic forces in the E-W walls

Seismic forces in the E-W walls can be determined in a similar manner. Since all walls in the E-W direction have the same geometry (height, length, thickness) and mechanical properties and consequently the same stiffness, the shear force will be equal at the East and West side. The force per side is equal to

$0.5V = 0.5 * 860 = 430$ kN

- Seismic forces in the E-W walls – torsional effects ignored

Shear force in each E-W wall is equal to (there are seven walls per side):

$$V_V = \frac{0.5V}{7} = \frac{430}{7} = 61 \text{ kN}$$

- Seismic forces in the E-W walls – torsional effects considered:

$$V_W = \frac{V}{7} * 0.6 = \frac{860}{7} * 0.6 = 74 \text{ kN}$$

6. Check whether the structure is torsionally sensitive (see Section 1.5.9.2).

NBCC 2005 Cl. 4.1.8.11.(9) requires that the torsional sensitivity B of the structure be determined by comparing the maximum horizontal displacement anywhere on a storey, to the average displacement of that storey. Torsional sensitivity is determined in a similar manner as the effect of accidental torsion, that is, by applying a set of a set of lateral forces at a distance of $\pm 0.1 D_{nx}$ from the centre of mass C_M. In case of a rigid diaphragm, displacements are proportional to the forces developed in the walls. Therefore, B can be determined by comparing the forces at the sides of the building with/without the effect of accidental torsion.

The maximum displacement would be proportional to 0.6V, while the displacement on the other side would be proportional to 0.4V. Thus the average displacement is proportional to 0.5V. Thus

$$B = \frac{0.6V}{0.5V} = 1.2$$

Since $B < 1.7$, this building is not torsionally sensitive and the equivalent static analysis would have also been allowed under criterion b) as discussed in step 4 above.

7. Discussion

It was assumed at the beginning of this example that the roof structure can be modeled like a rigid diaphragm. If this roof was modeled like a flexible diaphragm, the shear forces in each N-S wall would be equal to 0.5V. From a reliability point of view, it does not seem quite right that the forces are smaller for a flexible diaphragm than a rigid one - it should be the other way around. On the other hand, the flexible diaphragm may have a longer period and the forces would be smaller (see Example 3 for a detailed discussion on rigid and flexible diaphragm models).

<u>EXAMPLE 2</u>: Seismic load calculation for a medium-rise masonry building to NBCC 2005

A typical floor plan and vertical elevation are shown below for a four-storey mixed use (commercial/residential) building located near the intersection of Granville Street and 41st Avenue in Vancouver, BC. The ground floor is commercial with a reinforced concrete slab separating it from the residential floors, which have lighter floor system consisting of steel joists supporting a composite steel and concrete deck. The front of the building is mostly glazing, which has no structural application.

First, determine the seismic force for this building according to the NBCC 2005 equivalent static force procedure, and a vertical force distribution in the E-W direction. Find the base shear and overturning moment in the E-W walls. Assume that the floors act as rigid diaphragms and that the strong N-S walls can resist the torsion.

Next, consider the torsional effects in all walls and find the forces in the E-W walls. Compare the seismic forces obtained with and without torsional effects.

For the purpose of weight calculations, use 200 mm blocks for N-S walls and 300 mm blocks for E-W walls. All walls are solid grouted (this is a conservative assumption appropriate for a preliminary design) and the compressive strength f'_m is 10.0 MPa. Grade 400 steel has been used for the reinforcement. The building is of normal importance and is supported on Class C soil. Consider "limited ductility" reinforced masonry shear walls.

Movement joints are not to be considered in this example. Note that movement joints in the N-S walls would have caused slight changes in the stiffness values of these walls.

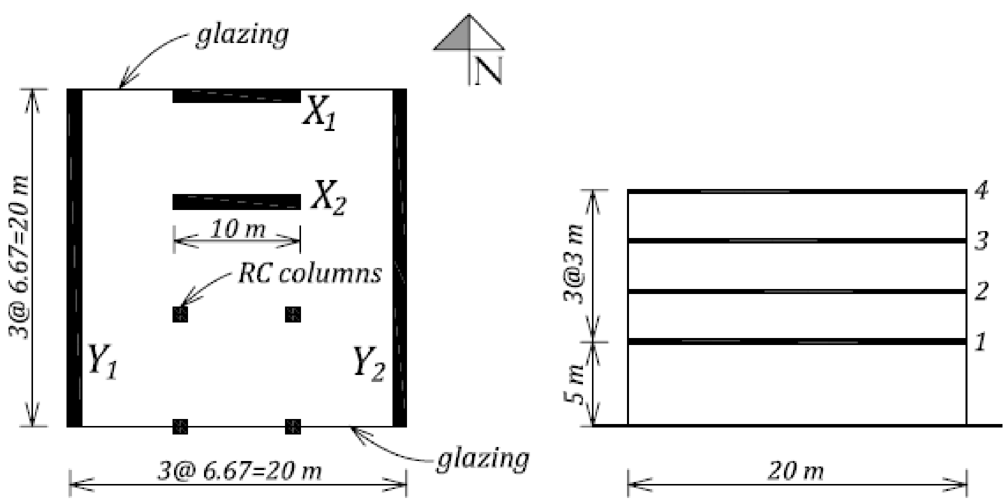
Specified loads (note that roof and floor loads include a 1 kPa allowance for partition walls and glazing):

4th floor (roof level) = 3 kPa — Note: 1 kPa = 1 kN/m^2

2nd and 3rd floor = 4 kPa

1st floor (concrete floor) = 6 kPa

25% snow load = 0.4 kPa



Plan *Elevation*

SOLUTION:

1. Design assumptions

- Rigid diaphragm
- All walls are solid grouted

2. Calculate the seismic weight W (see Table 1-12 and NBCC 2005 Cl.4.1.8.2)

Wall weight:

N-S walls - 200 mm thick w = 4.18 kPa

E-W walls – 300 mm thick w = 6.38 kPa

Note that, for the purpose of seismic weight calculations, the length of a N-S wall is 20 m, while the length of an E-W wall is 10.0 m.

Seismic weight W_1:

$$W_1 = \left(\frac{5.0m}{2} + \frac{3.0m}{2}\right)(4.18kPa * 2 * 20m + 6.38kPa * 2 * 10.0m) + (6.0kPa)(20m * 20m) = 3579kN$$

Seismic weight W_2:

$$W_2 = \left(\frac{3.0m}{2} + \frac{3.0m}{2}\right)(4.18kPa * 2 * 20m + 6.38kPa * 2 * 10.0m) + (4.0kPa)(20m * 20m) = 2484kN$$

Seismic weight W_3 (same as W_2) :

$$W_3 = 2484kN$$

Seismic weight W_4:

$$W_4 = \left(\frac{3.0m}{2}\right)(4.18kPa * 2 * 20m + 6.38kPa * 2 * 10.0m) + (3.0kPa + 0.4kPa)(20m * 20m) = 1802kN$$

Note that the seismic weight for each floor level is the sum of the wall weights and the floor weight. 25% snow load was included in the roof weight calculation. One-half of the wall height (below and above a certain floor level) was considered in the wall area calculations.

The total seismic weight is equal to

$$W = W_1 + W_2 + W_3 + W_4 = 3579 + 2484 + 2484 + 1802 \cong 10350kN$$

3. Calculate the seismic base shear force (see Section 1.5.4).

a) Find seismic design parameters used to determine seismic base shear.

- Location: Vancouver, BC (Granville and 41st Avenue)
  - $S_a(0.2)$ = 0.95 (NBCC 2005 Appendix C, page C-13)
  - $S_a(0.5)$ = 0.65
  - $S_a(1.0)$ = 0.34
  - $S_a(2.0)$ = 0.17
- Foundation factors
  Site Class C: F_a = 1.0 (Table 1-10 or NBCC 2005 Table 4.1.8.4.B) and F_v = 1.0 (Table 1-11 or NBCC 2005 Table 4.1.8.4.C)
- I_E = 1.0 (normal importance building)
- M_v = 1.0 (higher mode factor, equal to 1.0 for $T \leq 1.0$ sec)
- Building SFRS description: masonry structure – limited ductility shear walls for building height of 14 m (see Table 1-13 or NBCC 2005 Table 4.1.8.9), hence

R_d = 1.5 and R_o = 1.5

- Building period (T) calculation (NBCC 2005 Cl.4.1.8.11.3.c) – wall structures
  h_n = 14.0 m building height
  $T = 0.05(h_n)^{3/4}$ = 0.36 sec
- Site design spectrum $S(T)$ (see Section 1.5.2)

Since the soil is characterized as Site Class C,
$S(T) = S_a(T)$

- For T =0.2 sec:

$S(0.2) = F_a S_a(0.2)$= 1.0*0.95=0.95 $\boxed{S(0.2)=0.95}$

- For T =0.5 sec:

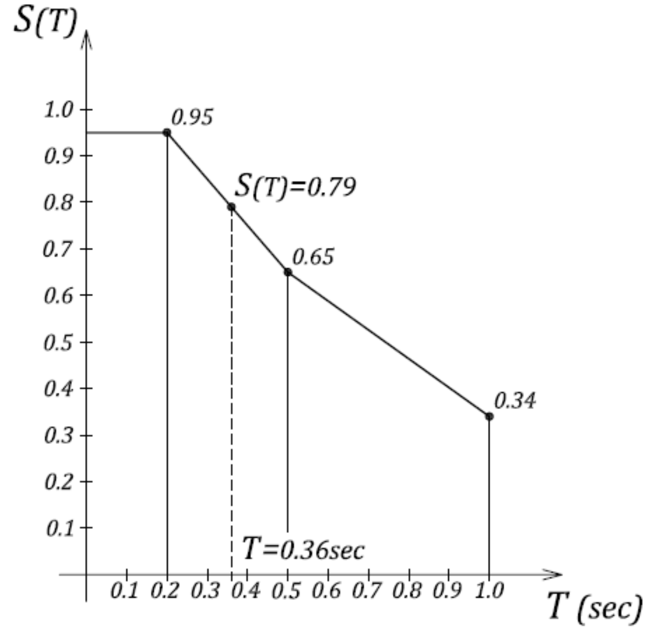
$S(0.5) = F_v S_a(0.5)$=1.0*0.65=0.65 $\boxed{S(0.5)=0.65}$
or (smaller value governs)
$S(0.5) = F_a S_a(0.2)$=1.0*0.95=0.95

- For T =1 sec:

$S(1.0) = F_v S_a(1.0)$=1.0*0.34=0.34 $\boxed{S(1.0)=0.34}$

- For T =2 sec:

$S(2.0) = F_v S_a(2.0)$= 1.0*0.17=0.17 $\boxed{S(2.0)=0.17}$



Building period T = 0.36 sec, so interpolate between $S(0.2)$ and $S(0.5)$, hence $\boxed{S(T)= 0.79}$

b) Compute the design base shear (NBCC 2005 Cl.4.1.8.11).

The design base shear V is determined according to the following equation:

$$V = \frac{S(T) M_v I_E}{R_d R_o} W = \frac{0.79*1.0*1.0}{1.5*1.5} W = 0.35W$$

Check the lower and upper bounds for the V value.

- Lower bound ($V_{\min}$) value (must be exceeded)

$$V_{\min} = \frac{S(2.0) M_v I_E W}{R_d R_o} = \frac{0.17*1.0*1.0}{1.5*1.5} W = 0.075W$$

- Upper bound ($V_{\max}$) value (base shear need not exceed this value)

$$V_{\max} = \left(\frac{2S(0.2)}{3}\right)\left(\frac{I_E W}{R_d R_o}\right) = \left(\frac{2*0.95}{3}\right)\left(\frac{1.0}{1.5*1.5}\right) W = 0.28W < 0.35W \qquad \text{<= Governs}$$

Note that the upper bound base shear value can be used only when $R_d \geq 1.5$.

Therefore, the design seismic base shear is equal to
$V = 0.28W = 0.28*10350 = 2898 \approx 2900$ kN

4. Determine if the equivalent static procedure can be used (see Section 1.5.7 and NBCC 2005 Cl. 4.1.8.7).

According to the NBCC 2005, the dynamic method is the default method, but the equivalent static method can be used if the structure meets any of the following criteria:

(a) is located in a region of low seismic activity where $I_E F_a S_a(0.2) < 0.35$***,***

In this case, seismic hazard index $I_E F_a S_a(0.2)$=1.0*1.0*0.95=0.95 > 0.35 and so this criterion is not satisfied.

(b) is a regular structure less than 60 m in height with period T ***< 2 seconds in either direction,***

This building is clearly less than 60 m in height and the period T < 2 sec (as discussed above). To confirm that this structure is regular, the designer needs to review the irregularities discussed in Section 1.5.10.1. It can be concluded that this building does not have any of the irregularity types identified by NBCC 2005 and so this criterion is satisfied.

(c) has any type of irregularity (other than Type 7 that requires the dynamic method if B >1.7), but is less than 20 m in height with period T ***< 0.5 seconds in either direction***

This is an irregular structure, but it is less than 20 m in height and the period is less than 0.5 sec. The torsional sensitivity B should be checked to confirm that B < 1.7 (see Section 1.5.9.2).

Since the criterion b) has been satisfied, the design can proceed by using the equivalent static analysis procedure.

5. Seismic force distribution over the building height (see Section 1.5.7).

According to NBCC 2005 Cl. 4.1.8.11.(6), the total lateral seismic force, V , is to be distributed over the building height in accordance with the following formula (see Figure 1-15):

$$F_x = (V - F_t) \cdot \frac{W_x h_x}{\sum_{i=1}^{n} W_i h_i}$$

where

F_x – seismic force acting at level x

F_t – a portion of the base shear to be applied in addition to force F_n at the top of the building.

In this case, F_t = 0 since the fundamental period is less than 0.7 sec.

Interstorey shear force at level x can be calculated as follows:

$$V_x = F_t + \sum_{x}^{n} F_i$$

Bending moment at level x can be calculated as follows:

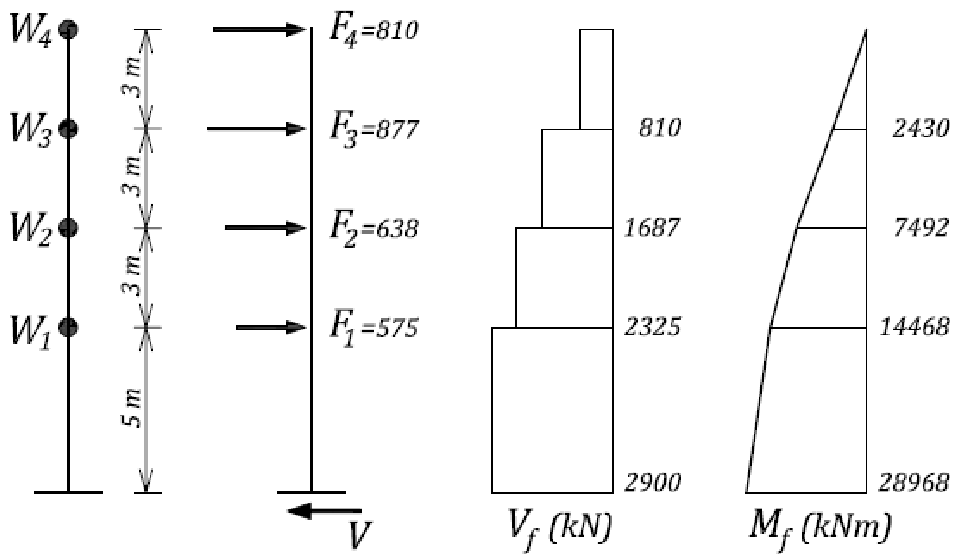
$$M_x = \sum_{i=x}^{n} F_i (h_i - h_x)$$

These calculations are presented in Table 1.

Table 1. Distribution of Seismic Forces over the Wall Height

Level	h_x (m)	W_x (kN)	$W_x h_x$	F_x (kN)	V_x (kN)	M_x (kNm)
4	14.0	1802	25228	810	810	0
3	11.0	2484	27324	877	1687	2430
2	8.0	2484	19872	638	2325	7492
1	5.0	3579	17895	575	2900	14468
$\sum$		10349	90319	2900		28968

Distribution of seismic forces over the building height and the corresponding shear and moment diagrams are shown on the figure below.



It is important to confirm that the sum of seismic forces F_x over the building height is equal to the base shear

$V_b = V =$ 2900 kN

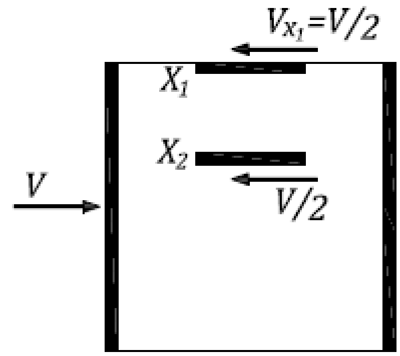
The bending moment at the base of the building, also called the base bending moment, is equal to

M_b = 28968 ≈ 29000 kNm.

6. Find the seismic forces in the E-W walls – torsional effects ignored.

Due to asymmetric layout of the E-W walls, the centre of mass C_M in the building under consideration does not coincide with the centre of resistance C_R, hence there are torsional effects in all walls. However, since the N-S walls are significantly more rigid compared to the E-W walls, it can be assumed that the N-S walls will resist the torsional effects (see step 8 for a detailed discussion). As a consequence, it can be assumed that the base shear force in the E-W direction is equally divided between the two E-W walls (see the figure on the next page), that is,

$$V_{xo} = \frac{V}{2} = \frac{2900}{2} = 1450 \text{ kN}$$



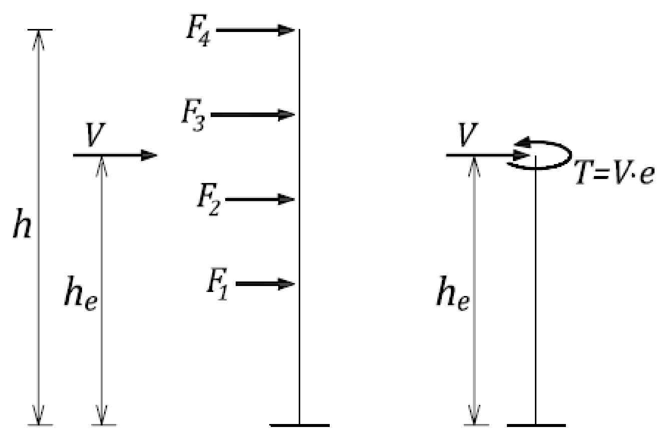
Similarly, the base bending moment in each wall is equal to

$$M_{bx} = \frac{M_b}{2} = \frac{29000}{2} = 14500 \text{ kNm}$$

7. Find the seismic forces in the E-W walls – torsional effects considered (see Section 1.5.9).

To determine the wall forces from the torsional forces a 3-D analysis should be made. Even though the walls are considered uniform over the entire height, the contribution of shear deformation relative to bending deformation is different over the height. An approximate method that does not require a 3-D analysis is to consider the structure as an equivalent single-storey structure. The entire shear is applied at the effective height, h_e, defined as the height at which the shear force V_f must be applied to produce the base moment M_f, that is,

$$h_e = \frac{M_f}{V_f} = \frac{29000}{2900} = 10.0 \text{ m}$$

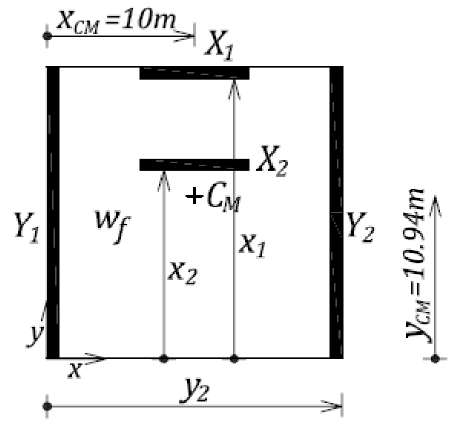


This model, although not strictly correct, will be used to determine the elastic distribution of the torsional forces as well as the displacements. The top displacement of the wall is assumed to be 1.5 times the displacement at the h_e height (see step 8 for displacement calculations).

Torsional moment (torque) is a product of the seismic force and the eccentricity between the centre of resistance (C_R) and the centre of mass (C_M), which will be calculated in the following tables.

First, the centre of mass will be determined, as shown on the figure on the next page. The calculations are summarized in Table 2.

Table 2. Calculation of the Centre of Mass (C_M)



Wall	w_i (kN)	x_i (m)	y_i (m)	$w_i * x_i$	$w_i * y_i$
X_1	733.7	10.00	20.00	7337	14674
X_2	733.7	10.00	13.33	7337	9780
Y_1	961.4	0	10.00	0	9614
Y_2	961.4	20.00	10.00	19228	9614
Floors	6960	10.00	10.00	69600	69600
$\sum$	10350			103502	113282

The C_M coordinates can be determined as follows:

$$x_{CM} = \frac{\sum_i w_i * x_i}{\sum_i w_i} = \frac{103502}{10350} = 10.00 \text{ m} \qquad y_{CM} = \frac{\sum_i w_i * y_i}{\sum_i w_i} = \frac{113282}{10350} = 10.94 \text{ m}$$

Next, the centre of resistance (C_R) will be determined, and the calculations are presented in Table 3, although because there are only two equal walls in each direction the C_R will lie between the walls.

Table 3. Calculation of the Centre of Resistance (C_R)

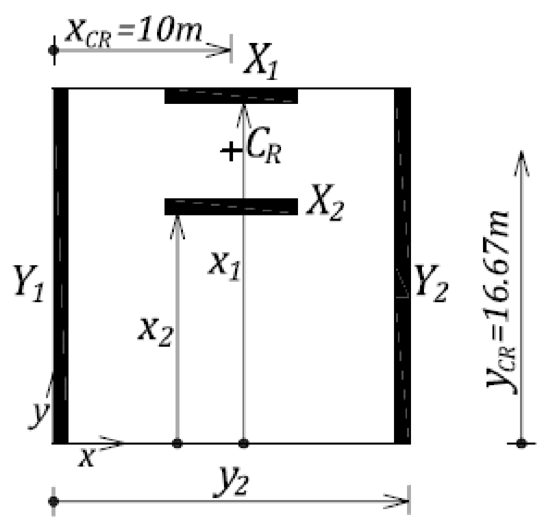
Wall	t (m)	h/l_w *	$K/(E_m \cdot t)$ **	K_x x10[3] (kN/m)	K_y x10[3] (kN/m)	x_i (m)	y_i (m)	$K_y \cdot x_i$ *10[3]	$K_x \cdot y_i$ *10[3]
X_1	0.29	1.0	0.143	352.5			20.00		7050.0
X_2	0.29	1.0	0.143	352.5			13.33		4699.0
Y_1	0.19	0.5	0.5		807.5	0		0	
Y_2	0.19	0.5	0.5		807.5	20.00		16150.0	
$\sum$				705.0	1615.0			16150.0	11750.0

Notes:

* - $h = h_e$ = 10.0 m effective wall height

** - see Table D-3

Note that the elastic uncracked wall stiffnesses K for individual walls have been determined from Table D-3, by entering appropriate height-to-length ratios. In this design, all walls and piers have been modelled as cantilevers (fixed at the base and free at the top) – see Section C.3 for more details regarding wall stiffness



calculations. The modulus of elasticity for masonry is $E_m = 8.5*10^6$ kPa (corresponding to f'_m of 10 MPa).

The C_R coordinates can be determined as follows (see the figure):

$$x_{CR} = \frac{\sum_i K_{yi} * x_i}{\sum_i K_{yi}} = \frac{16150*10^3}{1615*10^3} = 10 \text{ m}$$

$$y_{CR} = \frac{\sum_i K_{xi} * y_i}{\sum_i K_{xi}} = \frac{11750*10^3}{705*10^3} = 16.67 \text{ m}$$

Next, the eccentricity needs to be determined. Since we are looking for the forces in the E-W walls, we need to determine the actual eccentricity in the y direction (e_y), that is,

$e_y = y_{CR} - y_{CM} = 16.67 - 10.94 = 5.73$ m

In addition, the accidental eccentricity needs to be considered, that is,

$e_a = \pm 0.1 D_{ny} = \pm 0.1 * 20 = \pm 2.0$ m

The total maximum eccentricity in the y-direction is equal to

$e_{ty1} = e_y + e_a = 5.73 + 2.0 = 7.73$ m

or

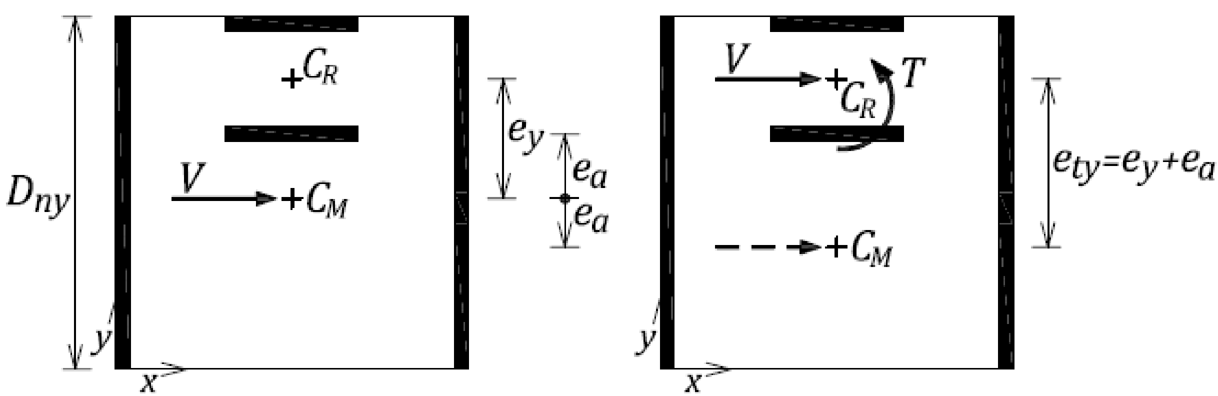
$e_{ty2} = e_y - e_a = 5.73 - 2.0 = 3.73$ m

Note that the latter value does not govern and will not be considered in further calculations.

Torsional moment is determined as a product of the shear force and the eccentricity, that is,

$T = V * e_{ty1} = 2900 * 7.73 = 22417$ kNm

Torsional effects are illustrated on the figure below.



Seismic force in each wall has two components: translational (no torsional effects) and torsional, that is,

$V_i = V_{io} + V_{it}$

where

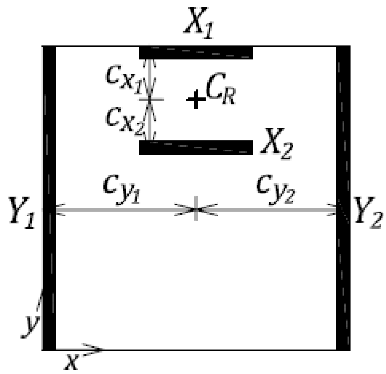
$V_{io} = V * \frac{K_i}{\sum K_i}$ translational component

and

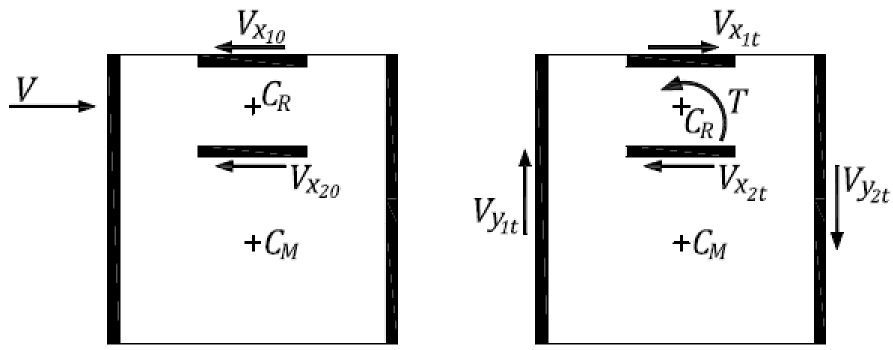
$V_{it} = \frac{T * c_i}{J} * K_i$ torsional component

$J = \sum K_{xi} \cdot c_{xi}^2 + \sum K_{yi} \cdot c_{yi}^2 = 169 * 10^6$ torsional stiffness (see Table 4)

c_{xi}, c_{yi} - distance of the wall centroid from the centre of resistance (C_R) (see the figure below)



Translational and torsional force components for the individual walls are shown below.



Translational Force Components

Torsional Force Components

Calculation of translational and torsional forces is presented in Table 4.

Table 4. Seismic Shear Forces in the Walls due to Seismic Load in the E-W Direction

Wall	K_x*10^3 (kN/m)	K_y*10^3 (kN/m)	c_i (m)	$\sum K_i \cdot c_i^2$*10^6	$\frac{K_x}{\sum K_x}$	V_{xo} (kN)	V_{xt} (kN)	V_{total} (kN)
X_1	352.5		-3.33	3.84	0.5	1450	-154	1296
X_2	352.5		3.33	3.84	0.5	1450	154	1604
Y_1		807.5	-10.00	80.80			-1070	-1070
Y_2		807.5	10.00	80.80			1070	1070
$\sum$	705.0	1615.0		169.0				

It can be concluded from the above table that the maximum force in the E-W direction is equal to 1604 kN. This is an increase of only 11% as compared to the total force of 1450 kN obtained ignoring torsional effects.

It can be noted that the contribution of E-W walls to the overall torsional moment T of 22417 kNm is not significant (see Table 4).

$T_{E-W} = 154kN*3.3m + 154kN*3.3m \cong 1017kNm$

because

$T_{E-W}/T = 1017/22417 = 0.045 \approx 5\%$

this shows that the E-W walls contribute only 5% to the overall torsional moment.

The contribution of N-S walls to the overall torsional moment is as follows:

$T_{N-S} = 1070kN*10m + 1070kN*10m = 21400kNm$

and

$T_{N-S}/T = 21400/22417 \approx 95\%$

and

$T = T_{E-W} + T_{N-S} = 1017 + 21400 \approx 22417kNm$ (this is also a check for the torsional forces)

Therefore, the assumption that the torsional effects are resisted by N-S walls only is reasonable, since these walls contribute approximately 95% to the overall torsional resistance.

8. Calculate the displacements at the roof level (consider torsional effects).

Approximate deflections in the E-W walls can be determined according to the procedure outlined below. It should be noted that the force distribution calculations have been performed using elastic wall stiffnesses obtained from Table D-3. It is expected that the walls are going to crack during earthquake ground shaking; this will cause a drop in the wall stiffnesses. For the purpose of deflection calculations, we are going to use a reduction in the elastic stiffness (K) value to account for the effect of cracking.

a) The reduced stiffness to account for the effect of cracking (see Section C.3.5)

The reduced stiffness for walls X_1 *and* X_2 will be determined according to equation (15) from Section C.3.5, that is,

$$K_{ce} = (\frac{100}{f_y} + \frac{P_f}{f'_m A_e})K_c$$

where

K_c is elastic uncracked stiffness

$P_f = (2*6.67*6.67)(3.0+2*4.0+6.0) = 1513$ kN (axial force due to dead load in wall X_2)

$A_e = (290*10^3)*10.0 = 290*10^4$ mm^2 (effective cross sectional area for 290 mm block wall, solid grouted, length 10.0 m; see Table D-1 for A_e values for the unit wall length)

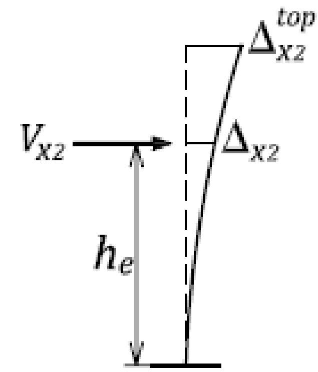
f'_m =10.0 MPa

$f_y = 400$ MPa (Grade 400 steel)

thus

$$K_{ce} = (\frac{100}{400} + \frac{1513*10^3}{10.0*290*10^4})K_c = 0.3K_c$$

b) The translational displacement in the walls X_1 and X_2 can be calculated as follows

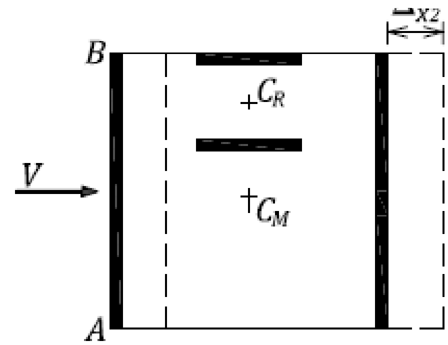


$$\Delta_{X20} = \frac{V_{X2o}}{0.3K_{X2}} = \frac{1450kN}{0.3*352.5*10^3 kN/m} = 13.7mm$$

According to NBCC 2005 Cl. 4.1.8.13, these deflections need to be multiplied by the $R_d R_o / I_E$ ratio (see Section 1.5.11). In this case, I_E = 1.0, and so

$$\Delta_{X20} = (13.7mm) R_d R_o = 13.7*1.5*1.5 = 30.8mm$$

Since the previous analysis assumed that the seismic force acts at the effective height h_e, the displacement at the top of the wall will be larger (see the figure). The top displacement can be calculated by deriving the displacement value at the tip of the cantilever; alternatively, an approximate factor of 1.5 can be used as follows:



$$\Delta_{X20}^{top} = 1.5*\Delta_{x2} = 1.5*30.8mm \approx 46mm$$

Since this is a rigid diaphragm, it can be assumed that the translational displacements are equal at a certain floor level – let us use point A at the South-East corner as a reference (see the figure).

c) The torsional displacements can be calculated as follows:

Torsional rotation of the building θ can be determined as follows, considering the reduced torsional stiffness to account for cracking (same as discussed in step a) above):

$$\theta = \frac{T}{J} = \frac{22417kNm}{0.3*169*10^6} = 4.421*10^{-4} \text{ rad}$$

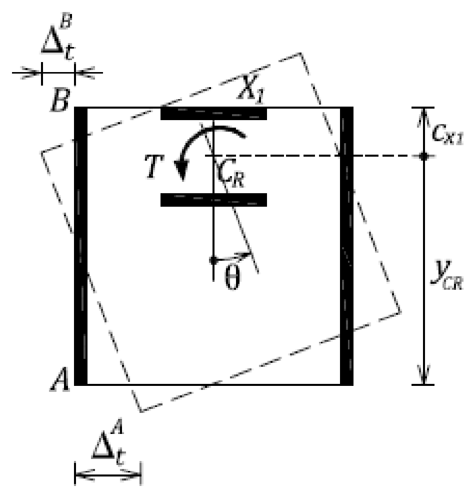
where (see the step 7 calculations)

$T = 22417$ kNm torsional moment

$J = 169*10^6$ elastic torsional stiffness

The maximum torsional displacement at the South-East corner in the X direction (see point A on the figure):

$$\Delta^A{}_t = \theta * Y_{CR} = 4.421*10^{-4}*16.67m = 7.4mm$$



Similarly as above, these displacements need to be multiplied by $R_d R_o / I_E$ and also by 1.5 to determine the displacement at the top of the roof, and so

$$\Delta^A{}_t{}^{top} = 1.5*7.4*R_d R_o \approx 25.0mm$$

d) Finally, the total maximum displacement at the roof level (at point A) is equal to:

$$\Delta^A{}_{\max} = \Delta_{X2}{}^{top} + \Delta^A{}_t{}^{top} = 46 + 25 = 71mm$$

9. Check whether the building is torsionally sensitive.

NBCC 2005 Cl. 4.1.8.11.(9) requires that the torsional sensitivity B of the structure be determined by comparing the maximum horizontal displacement anywhere on a storey to the average displacement of that storey (see Section 1.5.9.2 and Figure 1-19). This should be done for every storey, but in this case will only be done for the one storey as the remaining storeys will have similar B values because of the vertical uniformity of the walls. Torsional sensitivity is determined in a similar manner like the effect of accidental torsion, that is, by applying a set of lateral forces at a distance of $\pm 0.1 D_{nx}$ from the centre of mass C_M. Since the purpose of this evaluation is to compare deflections at certain locations relative to one another, it is not critical to use cracked wall stiffnesses.

In this case, the total maximum displacement at point A was determined in step 8 above, that is, $\Delta^A{}_{\max} = 71mm$

We need to determine the displacement at other corner (point B), that is, the minimum displacement. This can be done as follows:

Translational component:

$$\Delta^B{}_0 = \Delta_{X20}{}^{top} = 46mm$$

Torsional component:

$$\Delta_t = \theta * c_{X1} = 4.421*10^{-4} * 3.3m \approx 1.5mm$$

These displacements need to be multiplied by $R_d R_o / I_E$ and also by 1.5 to determine the displacement at the top of the roof, and so

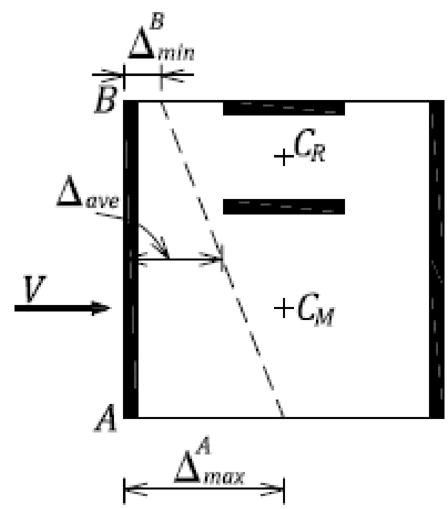
$$\Delta^B{}_t = 1.5*1.5*R_d R_o = 5mm$$

Since the direction of torsional displacements is opposite from the translational displacements, it follows that

$$\Delta^B{}_{\min} = \Delta^B{}_o - \Delta^B{}_t = 46 - 5 = 41mm$$

The average displacement at the roof level in the E-W direction (see the figure showing the displacement components):

$$\Delta_{ave} = \frac{\Delta^A{}_{\max} + \Delta^B{}_{\min}}{2} = \frac{71+41}{2} = 56mm$$



$$B = \frac{\Delta_{\max}}{\Delta_{ave}} = \frac{71.0}{56.0} = 1.27$$

Since B<1.7, this building is not considered to be torsionally sensitive. In general buildings with the main force resisting elements located around the exterior of the building will not be torsionally sensitive.

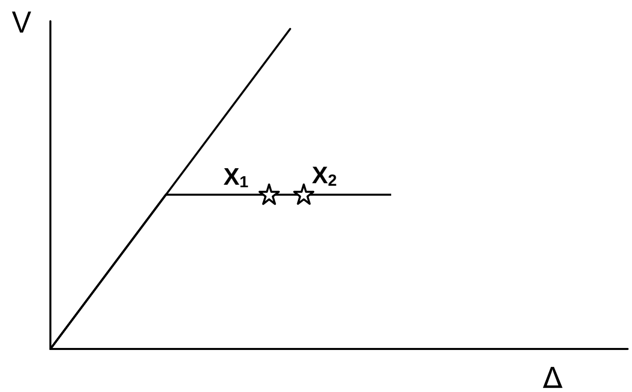
10. Discussion

A couple of important issues related to this design example will be discussed in this section.

a) Why should the N-S walls be considered to resist entire torsional effects?

The distribution of forces to the various elements in the structure is generally based on the relative elastic stiffnesses of the elements, unless the diaphragms are considered to be flexible and then the forces are distributed on the basis of contributory masses. The present example structure with four floors of concrete construction can be considered as having rigid diaphragms, and an elastic analysis was performed to determine the wall forces due to the torsional effects. Because the N-S walls are so much longer and stiffer than the E-W walls, and more widely separated, it is expected that they will resist most of the torque from the eccentricity. However, since we are designing the structures to respond inelastically, the distribution of forces from an elastic analysis should always be questioned. An argument is presented below to show that if the forces in the E-W walls are designed to be equal, they will not contribute to the torsional resistance.

The elastic torsional analysis for the forces in the E-W direction result in additional forces of ±154 kN in the E-W walls and ±1070 kN in the N-S walls (see Table 4). If all the torque is resisted by the N-S walls, the force in these walls would be ±1120 kN (an increase of only 50 kN).



For the earthquake load in the E-W direction the E-W walls must resist the total base shear in this direction and so they will have reached their yield strength and progressed along the flat portion of the shear/displacement curve as shown in the figure (assuming they have equal strength). The torsional load will have caused a small rotation of the diaphragms and so wall X_2 will have a slightly larger displacement than wall X_1, as shown on the figure. Had the walls remained elastic, the shear in wall X_2 would then be greater than wall X_1 and this would contribute to the torsional resistance. However in the nonlinear case, they both have the same shear resistance and so do not contribute to the torsional resistance. Thus in this example, all the torsion should be resisted by the longer N-S walls. The N-S walls are designed to resist the loads in the N-S direction but also to provide the torsional resistance from the loads in the E-W direction. However, it is highly unlikely that the maximum forces in the N-S walls from the two directions would occur at the same time, and practice has been to consider only 30% of the loads in one direction when combining with the loads in the other direction. Thus the forces in the N-S walls at the time of the maximum torsional forces from the N-S direction could reach the yield level on one side, but the torsional displacement on the other side would be in the opposite direction, so the wall force would be much reduced in the other direction. The two N-S forces then provide a torque to resist the torsional motion. Although this resisting torque may not be as large as the elastic analysis would predict, the result would not be failure, but only slightly larger torsional displacements.

b) Application of the "100%+30%" rule

In the calculation of total wall seismic forces including the torsional effects (see step 7 above), the effect of seismic loads in E-W direction only was taken into consideration when calculating the forces in E-W walls. However, it is a good practice to consider the "100+30%" rule that requires the forces in any element that arise from 100% of the loads in one direction be combined with 30% of the loads in the orthogonal direction (for more details refer to NBCC 4.1.8.8.(1)c and the commentary portion in Section 1.5.9.3 of this document).

Let us determine the forces in one of the E-W walls, e.g. wall X_2, by applying the "100+30%"rule. If only 100% of the force in the E-W direction is considered, the total force in the wall is equal to (see Table 4):

$$V_{X2}^{E-W} = V_{X2o} + V_{X2t} = 1450 + 154 = 1604 kN$$

If the seismic load is applied in the N-S direction, the torsional moment would be determined based on the accidental eccentricity e_a (since the building is symmetrical in that direction), and so the torsional force in the wall X_2 can be prorated by the ratio of torsional eccentricities in the E-W and N-S directions as follows,

$$V_{X2}^{N-S} = V_{X2t} * \frac{e_a}{e_y} = 154 * \frac{2.0m}{7.73m} = 39.8 \approx 40kN$$

The total seismic force in the wall X_2 due to 100% of the load in E-W direction and 30% of the load in the N-S direction can be determined as

$$V_{X2} = V_{X2}^{E-W} + 0.3V_{X2}^{N-S} = 1604 + 0.3 * 40 = 1616kN$$

It can be concluded that the difference between the force of 1616 kN (when the “100+30%” rule is applied) and the force of 1604 kN (when the rule is ignored) is insignificant.

However, it can be shown that the “100+30%” rule would significantly influence the forces in the N-S walls. When the seismic force acts in the E-W direction, the force in the N-S wall (e.g. wall Y_1) due to torsional effects is equal to (see Table 4)

$$V_{Y1}^{E-W} = 1070kN$$

When the seismic force acts in the N-S direction, the total force in the wall Y_1 (including the effect of accidental torsion) can be determined as (see Example 1 for a detailed discussion on accidental torsion)

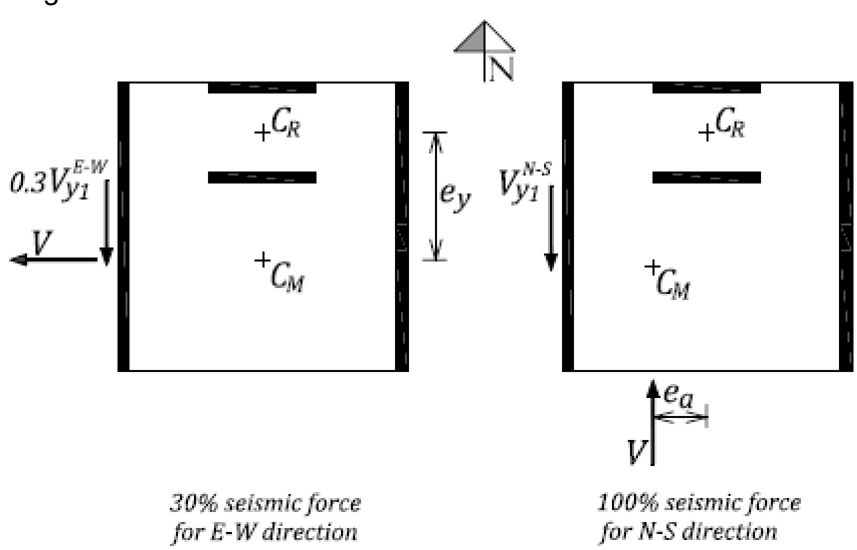
$$V_{Y1}^{N-S} = 0.6 * V = 0.6 * 2900 = 1740kN$$

So, if we apply the “100+30%” rule to 100% of the force in the N-S direction and 30% of the force in the E-W direction the resulting total force is equal to

$$V_{Y1} = V_{Y1}^{N-S} + 0.3V_{Y1}^{E-W} = 1740 + 0.3 * 1070 = 2061kN$$

In this case, it can be concluded that the difference between the force of 2061 kN (when the “100+30%” rule is applied) and the force of 1740 kN (when the rule is ignored) is significant (around 18%). This is illustrated on the figure below.

For those cases where there is a large eccentricity in one direction and the torsional forces are mainly resisted by elements in the other direction, the contribution from the “100+30%” rule can be significant.



<u>EXAMPLE 3</u>: Seismic load distribution in a masonry building considering both rigid and flexible diaphragm alternatives

Consider a single-storey commercial building located in Nanaimo, BC on a Class C site. The building plan and relevant elevations are shown on the figure below. The building has an open north-west façade consisting mostly of glazing. The roof elevation is at 4.8 m above the foundation. The roof structure is supported by 240 mm reinforced block masonry walls and steel columns on the north-west side. Masonry properties should be determined based on 20 MPa block strength and Type S mortar (use f'_m of 10.0 MPa). Grade 400 steel has been used for the reinforcement.

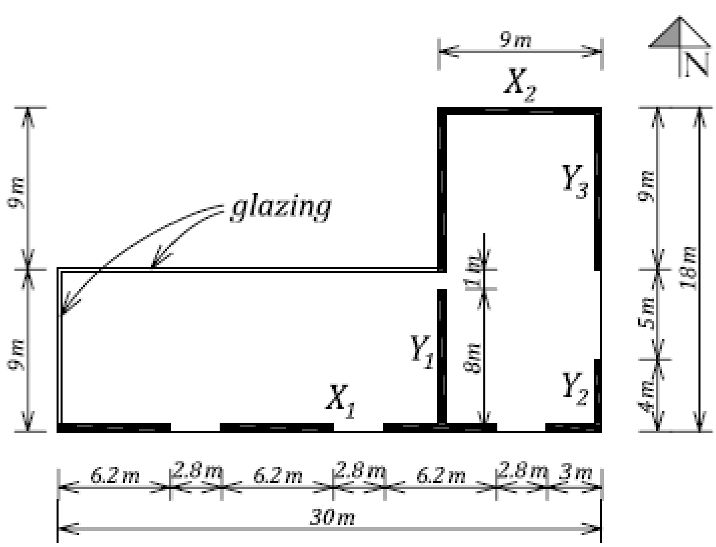
Masonry walls should be treated as "conventional construction" according to NBCC 2005 and CSA S304.1. A preliminary seismic design has shown that the total seismic base shear force for the building is equal to $V = 700$ kN. This force was determined based on the total seismic weight W of 2340 kN and the seismic coefficient equal to 0.3, that is, $V = 0.3W$.

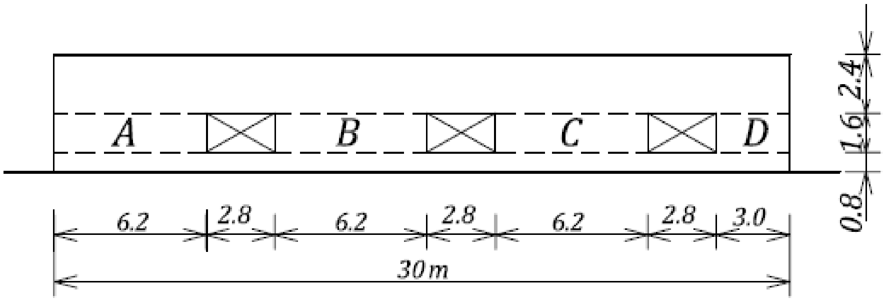
This example will determine the seismic forces in the N-S walls (Y_1 to Y_3) due to seismic force acting in the N-S direction for the following two cases:
a) Rigid roof diaphragm (consider torsional effects), and
b) Flexible roof diaphragm.

Finally, the wall forces obtained in parts a) and b) will be compared and the differences will be discussed.

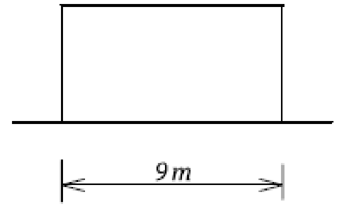
Note that both flexible and rigid diaphragms are considered to have the same weight, although this would be unlikely in a real design application. Also, the columns located on the north-west side are neglected in the seismic design calculations.

Specified loads:
roof = 3.5 kPa
25% snow load = 0.6 kPa
wall weight = 5.38 kPa (240 mm blocks solid grouted; this is a conservative assumption)

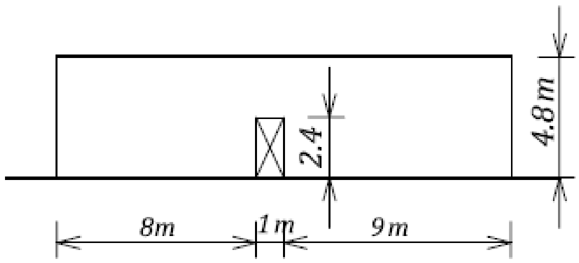




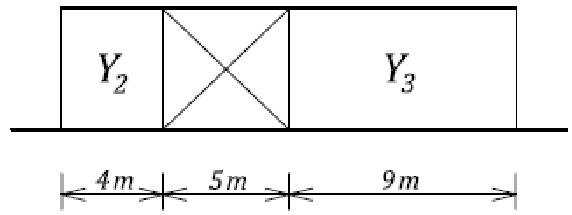
Wall X_1



Wall X_2



Wall Y_1



Walls Y_2 & Y_3

SOLUTION:

a) Rigid diaphragm

Torsional moment (torque) is a product of the seismic force and the eccentricity between the centre of resistance (C_R) and the centre of mass (C_M). The coordinates of the centre of mass will be determined taking into account the influence of wall masses, the upper half of which are supported laterally by the roof. The calculations are summarized in Table 1 below. Note that the centroid of the roof area is determined by dividing the roof plan into two rectangular sections.

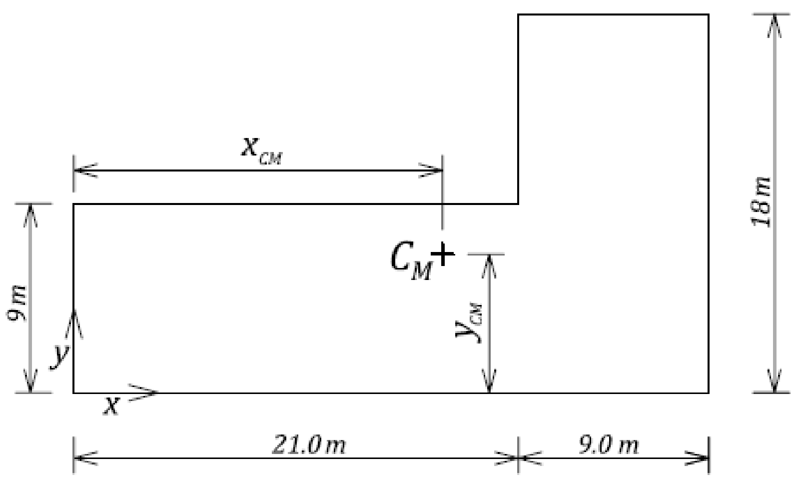
Table 1. Calculation of the Centre of Mass (C_M)

Wall	W_i (kN)	X_i (m)	Y_i (m)	$W_i * X_i$	$W_i * Y_i$
X1	387	15.00	0.00	5810	0
X2	116	25.50	18.00	2963	2092
Y1	232	21.00	9.00	4880	2092
Y2	52	30.00	2.00	1548	103
Y3	116	30.00	13.50	3486	1569
Roof 1	1107	15.00	4.50	16605	4982
Roof 2	332	25.50	13.50	8466	4482
	2343			43759	15319

The C_M coordinates have been determined from the table as follows (see the figure below):

$$x_{CM} = \frac{\sum_i W_i * X_i}{\sum_i W_i} = \frac{43757.02}{2343.86} = 18.68 \text{ m}$$

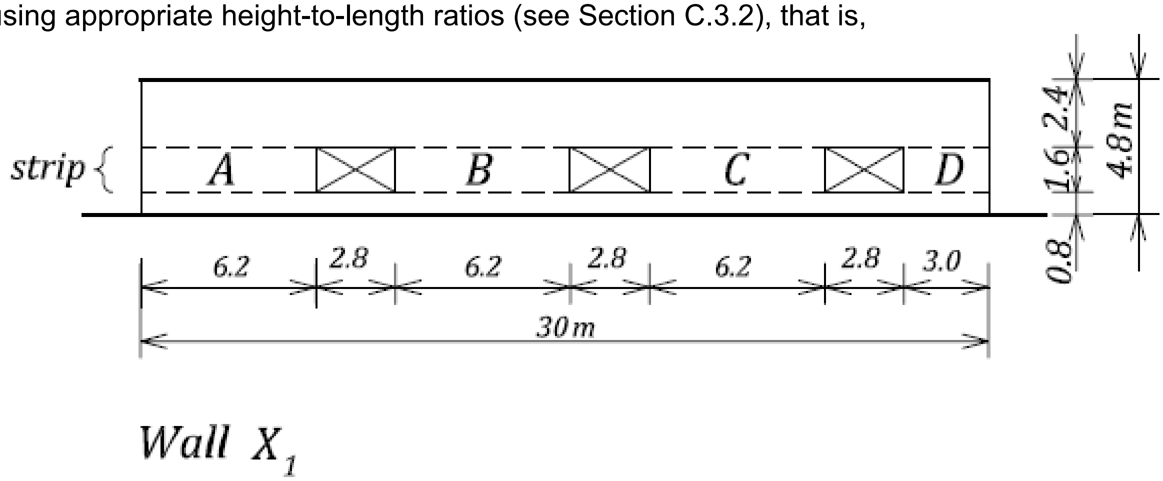
$$y_{CM} = \frac{\sum_i W_i * Y_i}{\sum_i W_i} = \frac{15324.38}{2343.86} = 6.54 \text{ m}$$



Next, the coordinates of the centre of resistance (C_R) will be determined. Wall X_1 has several openings and the overall wall stiffness is determined using the method explained in Section C.3.3 by considering the deflections of the following components for a unit load (see the figure on the next page):

- solid wall with 4.8 m height and 30 m length – cantilever (Δ_{solid})
- an interior strip with 1.6 m height (equal to the opening height) and 30 m length – cantilever (Δ_{strip})
- piers A, B, C, and D – cantilevered (Δ_{ABCD}) (the stiffness of the piers A, B, C, and D is summed and the inverse taken as Δ_{ABCD})

The stiffness of each component is based on the following equation for the cantilever model by using appropriate height-to-length ratios (see Section C.3.2), that is,



$$\frac{K}{E_m * t} = \frac{1}{\left(\frac{h}{l}\right)\left[4\left(\frac{h}{l}\right)^2 + 3\right]}$$

The overall wall deflection is determined from the combined pier deflections, as follows:

$$\Delta_{X1} = \Delta_{solid} - \Delta_{strip} + \Delta_{ABCD}$$

Note that the strip deflection is subtracted from the solid wall deflections - this removes the entire portion of the wall containing all the openings, which is then replaced with the deflection of the four piers.

Finally, the stiffness of the wall X_1 is equal to the reciprocal of the deflection (see Table 2), as follows

$$K_{X1} = \frac{1}{\Delta_{X1}} = 1.71$$

Table 2. Wall X_1 Stiffness Calculations

Wall	t (m)	h (m)	l (m)	**End conditions**	h/l	$K/(E*t)$	**Displacement**	$K_{final}/(E*t)$
Solid	0.24	4.8	30.0	cant	0.160	2.015	0.496	
Opening strip	0.24	1.6	30.0	cant	0.053	6.226	-0.161	
X1A	0.24	1.6	6.2	cant	0.258	1.186		
X1B	0.24	1.6	6.2	cant	0.258	1.186		
X1C	0.24	1.6	6.2	cant	0.258	1.186		
X1D	0.24	1.6	3.0	cant	0.533	0.453		
					$\sum$ (ABCD)	4.012	0.249	
							0.585	1.709

The stiffness of wall Y_1 is determined in the same manner (see the figure below). The calculations are summarized in Table 3.

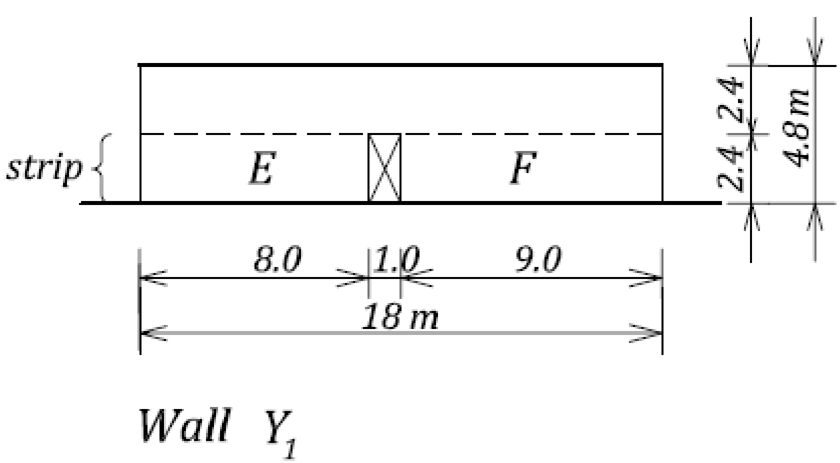


Table 3. Wall Y_1 Stiffness Calculations

Wall	t (m)	h (m)	l (m)	End conditions	h/l	$K/(E*t)$	Displacement	$K_{final}/(E*t)$
Solid	0.24	4.8	18	cant	0.267	1.142	0.876	
Opening strip	0.24	2.4	18	cant	0.133	2.442	-0.409	
Pier E	0.24	2.4	8	cant	0.300	0.992		
Pier F	0.24	2.4	9	cant	0.267	1.142		
					sum(EF)	2.134	0.469	
							0.935	1.070

Next, the centre of resistance (C_R) will be determined, and the calculations are presented in Table 4.

Table 4. Calculation of the Centre of Resistance (C_R)

Wall	t (m)	h (m)	l (m)	End cond.	h/l	$\frac{K}{E*t}$	K_x (kN/m)	K_y (kN/m)	X_i (m)	Y_i (m)	$K_y * X_i$	$K_x * Y_i$
X1	0.24					1.709*	3.49E+06	0	15	0		0.00E+00
X2	0.24	4.8	9	cant	0.53	0.453	9.24E+05	0	25.5	18		1.66E+07
Y1	0.24					1.070**	0	2.18E+06	21	0	4.58E+07	
Y2	0.24	4.8	4	cant	1.20	0.095	0	1.94E+05	30	0	5.82E+06	
Y3	0.24	4.8	9	cant	0.53	0.453	0	9.24E+05	30	0	2.77E+07	
							4.41E+06	3.30E+06			7.94E+07	1.66E+07

Notes:
* - see Table 2
** - see Table 3

Note that all walls and piers in this example were modeled as cantilevers (fixed at the base and free at the top). For more discussion related to modelling of masonry walls and piers for seismic loads see Section C.3. The modulus of elasticity for masonry is taken as $E_m = 8.5*10^6$ kPa (corresponding to f'_m of 10 MPa).

The C_R coordinates can be determined as follows (see the figure below):

$$x_{CR} = \frac{\sum_i K_{yi} * x_i}{\sum_i K_{yi}} = \frac{7.94*10^7}{3.30*10^6} = 24.05 \text{ m}$$

$$y_{CR} = \frac{\sum_i K_{xi} * y_i}{\sum_i K_{xi}} = \frac{1.66*10^7}{4.41*10^6} = 3.77 \text{ m}$$

Next, the eccentricity needs to be determined. Since we are considering the seismic load effects in the N-S direction, we need to determine the actual eccentricity in the x-direction (e_x), that is,

$$e_x = x_{CR} - x_{CM} = 24.05 - 18.68 = 5.37 \text{ m}$$

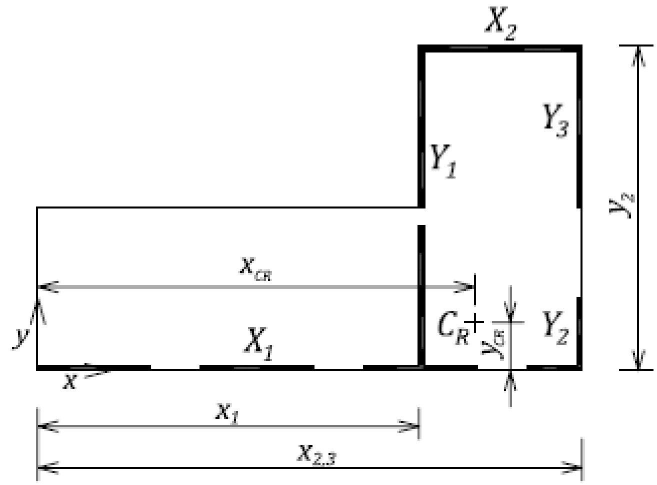
In addition, an accidental eccentricity needs to be considered, as follows:

$e_a = \pm 0.1 D_{nx} = \pm 0.1 * 30 = \pm 3.0$ m

The total maximum eccentricity in the x-direction assumes the following two values depending on the sign of the accidental eccentricity, that is,

$e_{x1} = e_x + e_a = 5.37 + 3.0 = 8.37$ m

$e_{x2} = e_x - e_a = 5.37 - 3.0 = 2.37$ m



The torsional moment is determined as a product of the shear force and the eccentricity, that is,

$T_1 = V * e_{x1} = 700 * 8.37 \approx 5860$ kNm

$T_2 = V * e_{x2} = 700 * 2.37 \approx 1660$ kNm

The seismic force in each wall can be determined as the sum of the two components: translational (no torsional effects) and torsional, that is,

$V_i = V_{io} + V_{it}$

where

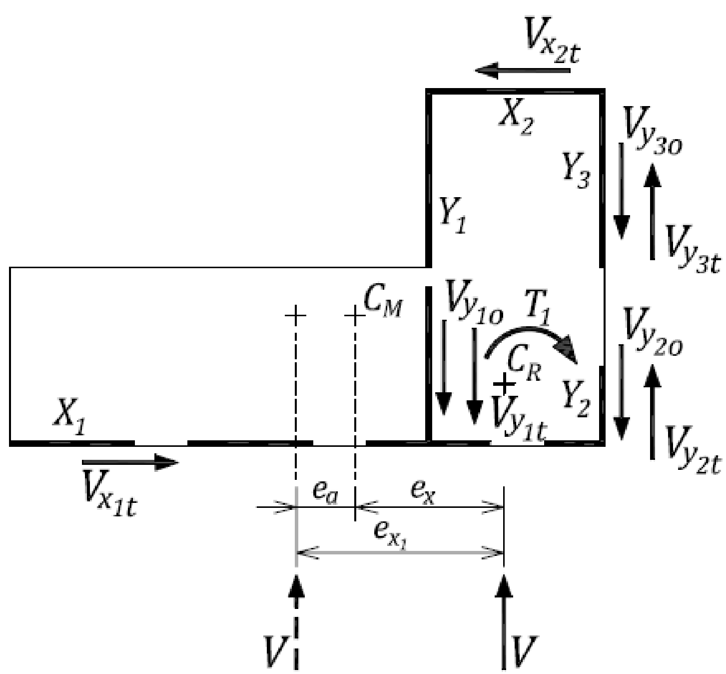
$V_{io} = V * \frac{K_i}{\sum K_i}$ translational component

$V_{it} = \frac{T * c_i}{J} * K_i$ torsional component

$J = \sum K_{xi} \cdot c_{xi}^2 + \sum K_{yi} \cdot c_{yi}^2 = 2.97 * 10^8$ torsional rigidity (see Table 5)

c_{xi}, c_{yi} - distance of the wall centroid from the centre of resistance (C_R)

The calculation of translational and torsional forces is presented in Table 5. Translational and torsional force components due to the eccentricity e_{x1} and the torsional moment T_1 are shown on the figure. Note that the torque T_1 causes rotation in the same direction like the force V (showed by the dashed line) around point C_R (this is illustrated on Figure 1-18). The wall forces shown on the diagram are in the directions to resist the shear V and torque T_1, thus on wall Y1 the translational



force and torsional force act in the same direction, while in walls Y2 and Y3 these forces act in the opposite direction. The calculation of the forces is presented in Table 5 where the sign convention has horizontal wall forces positive to the left and vertical forces positive down, resulting in negative values for the torsional forces in walls X1, Y2 and Y3.

Table 5. Seismic Shear Forces in the Walls due to Seismic Load in the N-S Direction

Wall	K_i (kN/m)	c_i (m)	$K_i * c_i^2$	$K_y / \sum K_y$	V_o (kN)	V_{1t} (kN)	V_{1total} (kN)	V_{2t} (kN)	V_{2total} (kN)	V_{govern} (kN)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
X1	3.49E+06	-3.77	4.96E+07			-260	-260	-74	-74	260
X2	9.24E+05	14.23	1.87E+08			260	260	74	74	260
$\sum K_x$	4.41E+06									
Y1	2.18E+06	3.05	2.03E+07	0.66	463	131	594	37	500	594
Y2	1.94E+05	-5.95	6.87E+06	0.06	41	-23	18	-6	35	35
Y3	9.24E+05	-5.95	3.27E+07	0.28	196	-109	87	-31	165	165
$\sum K_y$	3.30E+06			1.00	700					
		$\sum K_i * c_i^2$	2.97E+08							

It should be noted that there are two total seismic forces for each wall in the N-S direction (corresponding to torsional moments T_1 and T_2) – see columns (8) and (10) in Table 5. The governing force to be used for design is equal to the larger of these two forces, as shown in column (11) of Table 5. Note that, in some cases, torsional forces have a negative sign and cause a reduction in the total seismic force, like in the case of walls Y2 and Y3.

b) Flexible diaphragm

It is assumed in this example that flexible diaphragms are not capable of transferring significant torsional forces to the walls perpendicular to the direction of the inertia forces. Therefore, the wall forces are determined as diaphragm reactions, assuming that diaphragms D1 and D2 act as beams spanning between the walls, as shown on the figure below. The diaphragm loads include the inertia loads of the walls supported laterally by the diaphragm. The SFRS wall inertia forces are added to the forces supporting the diaphragms to get the total wall load. The seismic coefficient of 0.3 will be used in these calculations (as defined at the beginning of this example).

<u>Shear forces in the walls Y_{1a} and Y_2 (diaphragm D1):</u>

Seismic force in the diaphragm D1 is due to the roof seismic weight and the wall X_1 inertia load:

$$V_{D1} = 0.3*[(9m*30m)*(3.5kPa + 0.6kPa) + 2.4m*30m*5.38kPa] = 448kN$$

The diaphragm is considered as a beam with the reactions at the locations of walls Y_{1a} and Y_2, that is,

$$R_{Y1a} = 448kN*15m/9m = 747kN$$

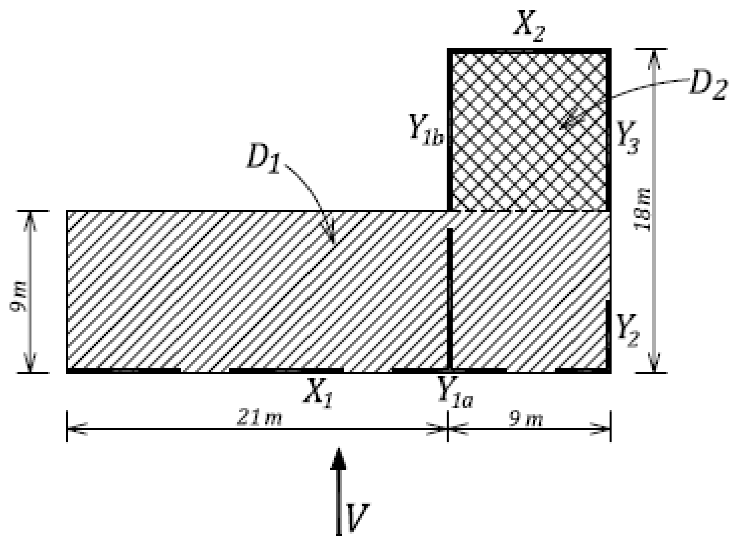
and

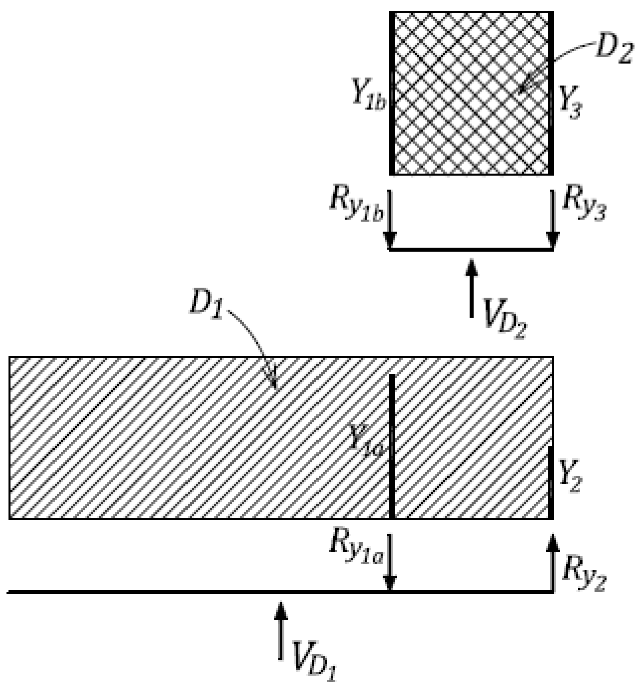
$R_{Y2} = V_{D1} - R_{Y1a} = 448 - 747 = -299kN$ (opposite direction from R_{Y1a} is required to satisfy equilibrium)

The total force in each wall is obtained when the wall inertia load is added to the diaphragm reaction, that is,

$V_{Y1a} = R_{Y1a} + V_w = 747 + 0.3 * 2.4m * 9m * 5.38kPa = 782kN$

$V_{Y2} = R_{Y2} + V_w = -299 + 0.3 * 2.4m * 4m * 5.38kPa = -284kN$ (note: this force has opposite direction from force V_{Y1a})





<u>Shear forces in the walls Y_{1b} and Y_3 (diaphragm D2):</u>

Seismic force in the diaphragm D2 is due to the roof seismic weight and the wall X_2 inertia load:

$V_{D2} = 0.3 * \left[(9m * 9m) * (3.5kPa + 0.6kPa) + 2.4m * 9m * 5.38kPa\right] = 134.5kN$

The diaphragm is considered as a beam with the reactions at the locations of walls Y_{1b} and Y_3, that is,

$R_{Y1b} = R_{Y3} = 134.5/2 = 67.3kN$

The total force in each wall is obtained when the wall inertia load is added to the diaphragm reaction, that is,

$V_{Y1b} = R_{Y1b} + V_w = 67 + 0.3 * 2.4m * 9m * 5.38kPa = 102kN$

$V_{Y3} = R_{Y3} + V_w = 67 + 0.3 * 2.4m * 9m * 5.38kPa = 102kN$

Total shear force in wall Y_1:

The total seismic force in the wall Y_1 is equal to

$V_{Y1} = V_{Y1a} + V_{Y1b} = 782 + 102 = 884kN$

Shear forces in walls Y_2 and Y_3:

The total shear force in the combined walls Y_2 and Y_3 is equal to

$V_{Y23} = V_{Y2} + V_{Y3} = -284 + 102 = -182kN$

This force will then be distributed to these walls in proportion to the wall stiffness, as follows (the wall stiffnesses are presented in Table 4):

$$V_{Y2} = \frac{K_{Y2}}{K_{Y2} + K_{Y3}} * V_{Y23} = \frac{1.94*10^5}{1.94*10^5 + 9.24*10^5} * (-182) = 0.17 * (-182) = -32kN$$

$V_{Y3} = V_{Y23} - V_{Y2} = -182 - (-32) = -150kN$

The comparison

Shear forces in the walls Y_1 to Y_3 obtained in parts a) and b) of this example are summarized on the figure below. A comparison of the shear forces is presented in Table 6.

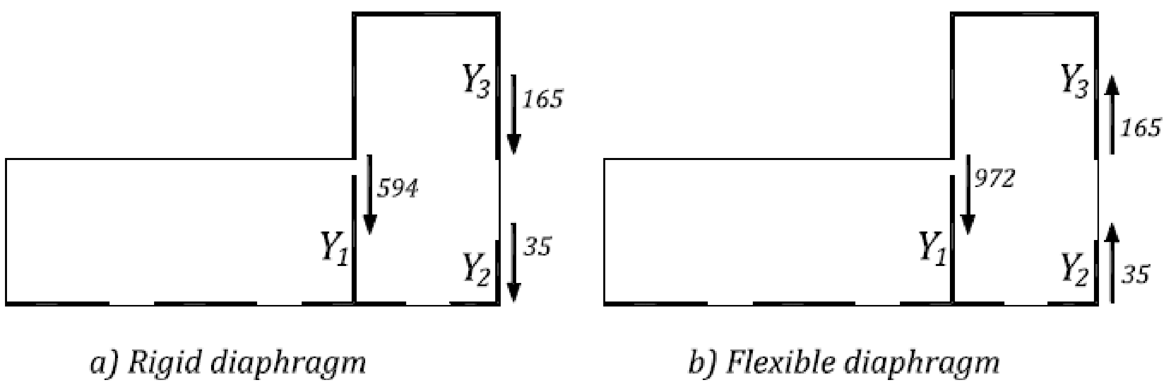


Table 6. Shear Forces in the Walls Y_1 to Y_3 for Rigid and Flexible Diaphragms

Wall	Shear forces (kN)	
	Rigid diaphragm (part a)	Flexible diaphragm (part b)
Y_1	594	972 (884)
Y_2	35	35 (32)
Y_3	165	165 (150)

Note that, for the flexible diaphragm case, values in the brackets are actual forces. These values are increased by 10 % to account for accidental eccentricity.

It can be observed from the table that the flexible diaphragm assumption results in the same seismic forces for the walls Y_2 and Y_3, and an increase in the wall Y_1 force.

Deflection calculations
A fundamental question related to diaphragm design is: when should a diaphragm be modeled as a rigid or a flexible one? This is discussed in Section 1.5.9.4. A possible way for comparing the extent of diaphragm flexibility is through deflections. The deflection calculations for the rigid and flexible diaphragm case are presented below.

- ***Rigid diaphragm (see Example 2, step 8 for a similar calculation)***

The deflection will be calculated for point A as this should be the maximum. First, a reduction in the wall stiffness to account for the effect of cracking will be determined following the approach presented in Section C.3.5. The reduced stiffness will be determined for wall Y_2 according to equation (15) from Section C.3.5, that is,

$$K_{ce} = (\frac{100}{f_y} + \frac{P_f}{f'_m A_e})K_c$$

where

K_c is the elastic uncracked stiffness

$P_f = 9.0*(9.0/2)*3.5 = 142$ kN (axial force due to dead load in wall X_2)

$A_e = (240*10^3)*9.0 = 216*10^4$ mm^2 (effective cross sectional area for 240 mm block wall, solid grouted, length 9.0 m; see Table D-1 for A_e values for the unit wall length)

f'_m =10.0 MPa

$f_y = 400$ MPa (Grade 400 steel)

thus

$$K_{ce} = (\frac{100}{400} + \frac{142*10^3}{10.0*216*10^4})K_c = 0.26K_c$$

Next, the translational displacement at point A can be calculated as follows:

$$\Delta^A{}_0 = \frac{V}{0.26\sum K_Y} = \frac{700kN}{0.26*3.3*10^6\,kN/m} = 0.82mm$$

Subsequently, the torsional displacement at point A will be determined. Torsional rotation of the building θ can be found from the following equation:

$$\theta = \frac{T}{J} = \frac{5860kNm}{0.26*297*10^6} = 7.59*10^{-5}\text{ rad}$$

where (see the torsional calculations performed in part a) of this example)

$T = 5860$ kNm torsional moment

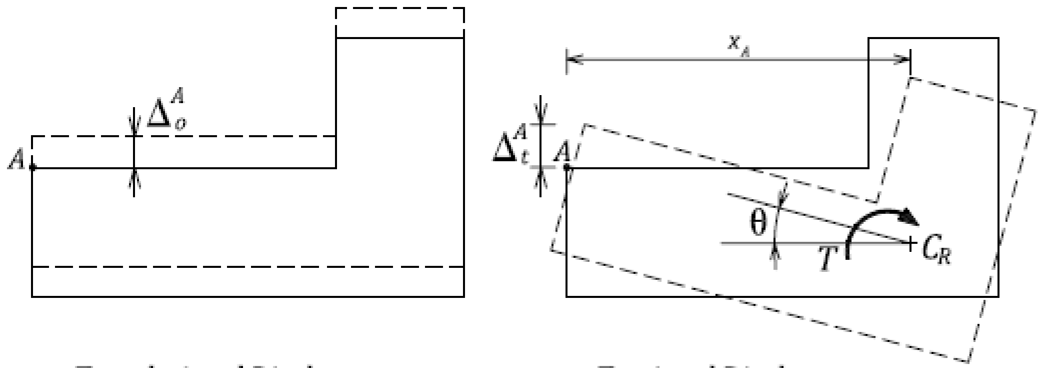
$J = 297*10^6$ elastic torsional stiffness (this value is reduced by 0.5 to take into account the cracking in the walls)

The torsional displacement at point A:

$$\Delta^A{}_t = \theta * x_A = 7.59*10^{-5}*24.05m = 1.82mm$$

The total displacement at point A is can be found as follows (note that the displacements need to be multiplied by $R_d R_o / I_E$ ratio, where I_E = 1.0):

$$\Delta^A{}_{\max} = \left(\Delta^A{}_0 + \Delta^A{}_t\right) * R_d R_o = \left(0.82 + 1.82\right) * 1.5 * 1.5 = 6.0mm$$



Translational Displacement *Torsional Displacement*

- **Flexible diaphragm**

As a first approximation the calculation will consider a 21 m long diaphragm portion as a cantilever beam subjected to the total shear force equal to:

$$V_D = 0.3 * \left[(9m * 21m) * (3.5kPa + 0.6kPa) + 2.4m * 21m * 5.38kPa\right] = 314kN$$

and the equivalent uniform load is equal to

$v_D = V_D / L = 15.0$ kN/m

where

$L = 21.0$ m diaphragm length for the cantilevered portion

The real deflection will be larger since the diaphragm acting as a cantilever is not fully fixed at the wall Y_1, and walls Y_1, Y_2, and Y_3 also deflect; both effects provide some rotation at the fixed end of the cantilever.

Consider a plywood diaphragm with the following properties:

$E = 1500$ MPa plywood modulus of elasticity

$G = 600$ MPa plywood shear modulus

$t_D = 25.4$ mm (1" plywood thickness)

$A = b * t_D = 9.0m * 0.0254m = 0.23$ m^2

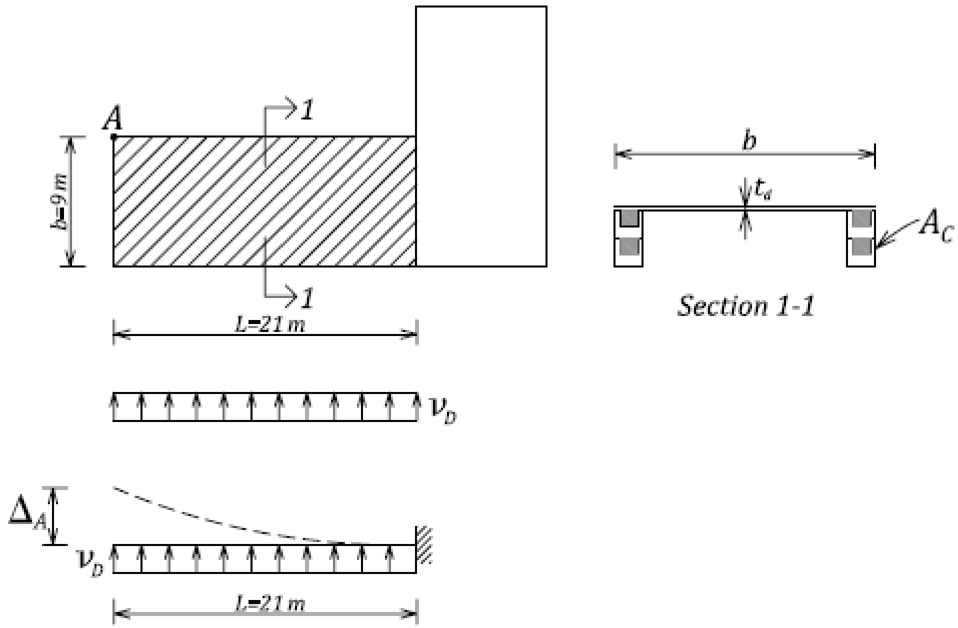
Let us assume that the two courses of grouted bond beam block act as a chord member, as shown on the figure. The roof-to-wall connection is achieved by means of nails driven into the anchor plate and hooked steel anchors welded to the plate embedded into the masonry. The corresponding moment of inertia around the centroid of the diaphragm can be found as follows:

$$I = 2 * A_c * \left(\frac{b}{2}\right)^2 = 2 * 0.096 * \left(\frac{9.0}{2}\right)^2 = 3.89 \text{ m}^4$$

where

$A_c = 2 * (0.24m * 0.2m) = 0.096$ m^2 chord area (two grouted 240 mm blocks)

$E_m = 8.5 * 10^6 kPa$ masonry modulus of elasticity based on f'_m = 10.0 MPa (solid grouted 20 MPa blocks and Type S mortar)



The total displacement at point A is equal to the combination of flexural and shear component, that is,

$$\Delta^A = \frac{v_D * L^4}{8E * I} + \frac{1.2V_D * L}{2 * A * G} = \frac{15.0 * (21.0)^4}{8 * 8.5 * 10^6 * 3.89} + \frac{1.2 * 314 * 21.0}{2 * 0.23 * 600 * 10^3} = (11.0 + 29.0) * 10^{-3} = 40 * 10^{-3} m = 40mm$$

The total displacement at point A is can be found by multiplying the above displacement by $R_d R_o / I_E$ ratio, that is,

$$\Delta^A{}_{\max} = \Delta^A * R_d R_o = 40 * 1.5 * 1.5 = 90mm$$

A quick check of the additional deflection caused by rotation at the fixed end of the cantilever indicates that an additional 50 mm could be expected at point A. Thus the total displacement would be about 140 mm.

By comparing the displacements for the rigid and flexible diaphragm model, it can be observed that the difference is significant:

$\Delta^A{}_{\max} = 6mm$ rigid diaphragm model

$\Delta^A{}_{\max} = 90mm$ flexible diaphragm model

Had the flexible diaphragm been used, the lateral drift ratio at point A would be equal to:

$$DR = \frac{\Delta_{\max}}{h_w} = \frac{90}{4800} = 0.019 = 1.9 \ \%$$

The drift is within the NBCC 2005 limit of 2.5% (see Section 1.5.11); however, a flexible diaphragm would not be an ideal solution for this design – a rigid diaphragm would be the preferred solution.

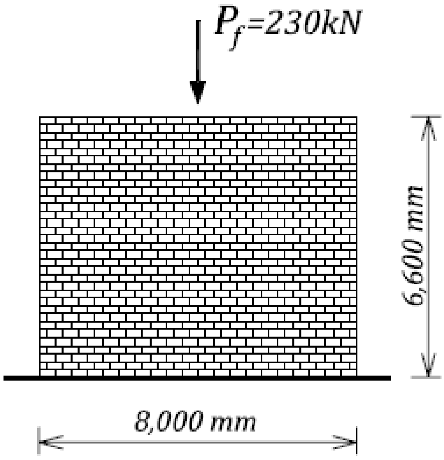
Discussion

In this example, seismic forces were determined for the N-S walls due to seismic load acting in the N-S direction. It should be noted, however, that there is a significant eccentricity causing torsional effects in the E-W walls due to seismic load acting in the E-W direction – these calculations were not included in this example.

EXAMPLE 4a: Minimum seismic reinforcement for a squat shear wall

Determine minimum seismic reinforcement according to CSA S304.1-04 for a loadbearing masonry shear wall located in an area with a seismic hazard index $I_E F_a S_a(0.2)$ of 0.66. The wall is subjected to axial dead load (including its own weight) of 230 kN.

Use 200 mm hollow concrete blocks of 15 MPa unit strength and Type S mortar. Grade 400 steel reinforcing bars (yield strength f_y = 400 MPa) and cold-drawn galvanized wire (ASWG) joint reinforcement are used for this design.



Wall dimensions:
l_w =8000 mm length
h_w = 6600 mm height
t = 190 mm thickness

SOLUTION:

The purpose of this example is to demonstrate how the minimum seismic reinforcement area should be determined and distributed in horizontal and vertical direction. Once the reinforcement has been selected in terms of its area and distribution, the flexural and shear resistance of the wall will be determined and the capacity design issues discussed, as well as the seismic safety implications of vertical and horizontal reinforcement distribution.

1. Material properties

Steel (both reinforcing bars and joint reinforcement):

f_y = 400 MPa ϕ_s = 0.85

Note that the cold-drawn galvanized wire has higher yield strength than Grade 400 steel, but it will be ignored for the small area included.

Masonry:

ϕ_m = 0.6

Assume partially grouted masonry. For 15MPa blocks and Type S mortar, it follows from Table 4 of S304.1-04 that

f'_m = 9.8 MPa

Based on Note 3 to Table 4, this f'_m value is normally used for hollow block masonry but can also be used for partially grouted masonry if the grouted area is not considered.

2. Find the minimum seismic reinforcement area and spacing (see Section 2.5.4.7 and Table 2-2).

Since $I_E F_a S_a(0.2)$=0.66 > 0.35, minimum seismic reinforcement must be provided (S304.1 Cl.10.15.2.2).

<u>Seismic reinforcement area</u>

Loadbearing walls, including shear walls, shall be reinforced horizontally and vertically with steel having a minimum area of

$A_{s\,\min} = 0.002 A_g$ = 0.002*(190*10^3 mm^2/m) = 380 mm^2/m

for 190 mm block walls, where

A_g =(1000mm)*(190mm)=190*10^3 mm^2/m gross cross-sectional area for a unit wall length of 1 m

Minimum area in each direction (one-third of the total area):

$A'_{h\,\min} = A'_{v\,\min} = 0.00067 A_g = \frac{A_{s\,\min}}{3} = \frac{380}{3} = 127$ mm^2/m

Thus the minimum total vertical reinforcement area

$A_{v\,\min} = 127 * l_w$ = (127 mm^2/m)(8 m) = 1016 mm^2

In distributing seismic reinforcement, the designer may be faced with the dilemma: should more reinforcement be placed in the vertical or in the horizontal direction? In theory, 1/3rd of the total amount of reinforcement can be placed in one direction and the remainder in the other direction. In this example, less reinforcement will be placed in the vertical direction, and more in the horizontal direction. The rationale for this decision will be explained later in this example.

<u>Vertical reinforcement (area and distribution) (see Table 2-2):</u>

According to S304.1 Cl.10.16.4.3.2, spacing of vertical reinforcing bars shall not exceed the <u>lesser of</u>:

- $6(t+10)$ =6(190+10)=1200 mm
- 1200 mm
- $l_w/4$ =8000/4=2000 mm

Therefore, the maximum permitted spacing of vertical reinforcement is equal to $s = 1200$ mm.

Since the maximum permitted bar spacing is 1200 mm, a minimum of 8 bars are required (note that the total wall length is 8000 mm). Therefore, let us use 8-15M bars, so

A_v = 8*200 =1600 mm^2

(note that the resulting reinforcement spacing is going to be less than 1200 mm, which is the upper limit prescribed by CSA S304.1).

The corresponding vertical reinforcement area per metre length is

$A'_v = \frac{A_v}{l_w} * 1000$ = 200 mm^2/m > $A'_{v\,\min}$ =127 mm^2/m   OK

Horizontal reinforcement (area and distribution) (see Table 2-2):
Let us consider a combination of joint reinforcement and bond beam reinforcement. According to S304.1 Cl.10.15.2.6, where both types of reinforcement are used, the maximum spacing of bond beams is 2400 mm and of joint reinforcement is 400 mm, so the following reinforcement arrangement is considered:

- 9 Ga. ladder reinforcement @ 400 mm spacing, and
- 2-15M bond beam reinforcement @ 2200 mm (1/3rd of the overall wall height). The area of ladder reinforcement (2 wires) is equal to 22.4mm^2, and the area of a 15M bar is 200 mm^2. So, the total area of horizontal reinforcement per metre of wall height is

$A'_h = \left(\frac{22.4}{400} + \frac{400}{2200}\right) * 1000 = 238$ mm^2/m > $A'_{h\min}$ =127 mm^2/m OK

So, the total area of horizontal and vertical reinforcement is

$A_s = A'_v + A'_h = 200 + 238$ =438 mm^2/m > $A_{s\min}$ =380 mm^2/m OK

Note that the total area (438 mm^2/m) exceeds the S304.1 minimum requirements (380 mm^2/m) by about 10%. It is difficult to select reinforcement that exactly meets the requirements, and also a reserve in reinforcement area provides additional safety for seismic effects.

3. Check whether the vertical reinforcement meets the minimum requirements for loadbearing walls (S304.1 Cl. 10.15.1.1 – see Table 2-2).
Since this is a shear wall, but also a loadbearing wall, pertinent reinforcement requirements would need to be checked, however the check is omitted from this example since it does not govern in seismic zones.

4. Determine the flexural resistance of the wall section (see Section C.1.1.2).
Design for combined effects of axial load and flexure will be performed by considering uniformly distributed vertical reinforcement. Based on the above discussion, the total area of vertical reinforcement is

A_{vt} = 1600 mm^2

At the base the wall is subjected to axial load P_f = 230 kN.

The in-plane moment resistance for the wall section can be determined approximately from the following equations:

$\alpha_1 = 0.85 \qquad \beta_1 = 0.8$

$$\omega = \frac{\phi_s f_y A_{vt}}{\phi_m f'_m l_w t} = \frac{0.85 * 400 * 1600}{0.6 * 9.8 * 8000 * 190} = 0.061$$

$$\alpha = \frac{P_f}{\phi_m f'_m l_w t} = \frac{230 * 10^3}{0.6 * 9.8 * 8000 * 190} = 0.026$$

$$c = \frac{\omega + \alpha}{2\omega + \alpha_1 \beta_1} l_w = \frac{0.061 + 0.026}{2 * 0.061 + 0.85 * 0.8}(8000) = 868 \text{ mm}$$ neutral axis depth

$$M_r = 0.5\phi_s f_y A_{vt} l_w \left(1 + \frac{P_f}{\phi_s f_y A_{vt}}\right)\left(1 - \frac{c}{l_w}\right) = 0.5 * 0.85 * \frac{400}{1000} * 1600 * \frac{8000}{1000}\left(1 + \frac{230 * 10^3}{0.85 * 400 * 1600}\right)\left(1 - \frac{868}{8000}\right)$$

$M_r = 2762$ kNm

5. Find the diagonal tension shear resistance (see Section 2.3.2 and CSA S304.1 Cl.10.10.1).

Find the masonry shear resistance (V_m):

$b_w = 190$ mm overall wall thickness

$d_v \approx 0.8 l_w = 6400$ mm effective wall depth

$\gamma_g = 0.5$ partially grouted wall

$P_d = 0.9 P_f$ = 207 kN

$v_m = 0.16(2 - \frac{M_f}{V_f d_v})\sqrt{f'_m}$ = 0.5 MPa

Note that a conservative assumption $\frac{M_f}{V_f d_v}$=1.0 has been made in the above equation.

$V_m = \phi_m (v_m b_w d_v + 0.25 P_d)\gamma_g$ = 0.6(0.5*190*6400+0.25*207*10^3)*0.5 = 198 kN

Steel shear resistance V_s:

$V_s = 0.6\phi_s \left(\sum A_v f_y \frac{d_v}{s}\right) = 0.6 * 0.85 * 608.8$ = 310 kN

where the shear reinforcement includes 9 Ga. joint reinforcement spaced at 400 mm, and 2-15M bond beam reinforcement at 2200 mm spacing, and so

$\sum A_v f_y \frac{d_v}{s} = \frac{22.4}{1000} * 400 * \frac{6400}{400} + \frac{400}{1000} * 400 * \frac{6400}{2200}$ = 608.8 kN

The total diagonal shear resistance is equal to

$V_r = V_m + V_s = 198 + 310 = 508$ kN

This is a squat shear wall because $\frac{h_w}{l_w} = \frac{6600}{8000} = 0.825 \le 1.0$.

Maximum shear allowed on the section is (S304.1 Cl.10.10.1.3):

$\max V_r = 0.4\phi_m \sqrt{f'_m} b_w d_v \gamma_g (2 - \frac{h_w}{l_w}) = 537$ kN

Since

$V_r < \max V_r$ OK

6. Sliding shear resistance (see Section 2.3.3)

The factored in-plane sliding shear resistance V_r is determined as follows.

μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

A_s = 1600 mm^2 total area of vertical wall reinforcement

$T_y = \phi_s A_s f_y$ = 0.85*1600*400 = 544 kN

P_d = 207 kN

$P_2 = P_d + T_y$ = 207+544 = 751 kN

$V_r = \phi_m \mu P_2$ = 0.6*1.0*751=451 kN

7. Capacity design check (see Section 2.5.2)

At this point, both the moment resistance M_r and the diagonal shear resistance V_r for the wall section have been determined. Seismic design philosophy considers that it is desirable to design structural members such that the more ductile flexural failure takes place before the more brittle shear failure has been initiated. This is known as capacity design approach and is discussed in detail in Section 2.5.2.

In this case, the factored moment resistance is equal to

$M_r = 2762$ kNm

The nominal moment resistance can be estimated as follows

$$M_n = \frac{M_r}{\phi_s} = \frac{2762}{0.85} = 3249 \text{ kNm}$$

Shear force at the top of the wall that would cause the overturning moment equal to M_n is equal to

$$V_{nb} = \frac{M_n}{h_w} = \frac{3249}{6.6} = 492 \text{ kN}$$

To ensure that flexural failure takes place before the diagonal shear failure, it is required that (see Figure 2-22)

$V_{nb} \le V_r$

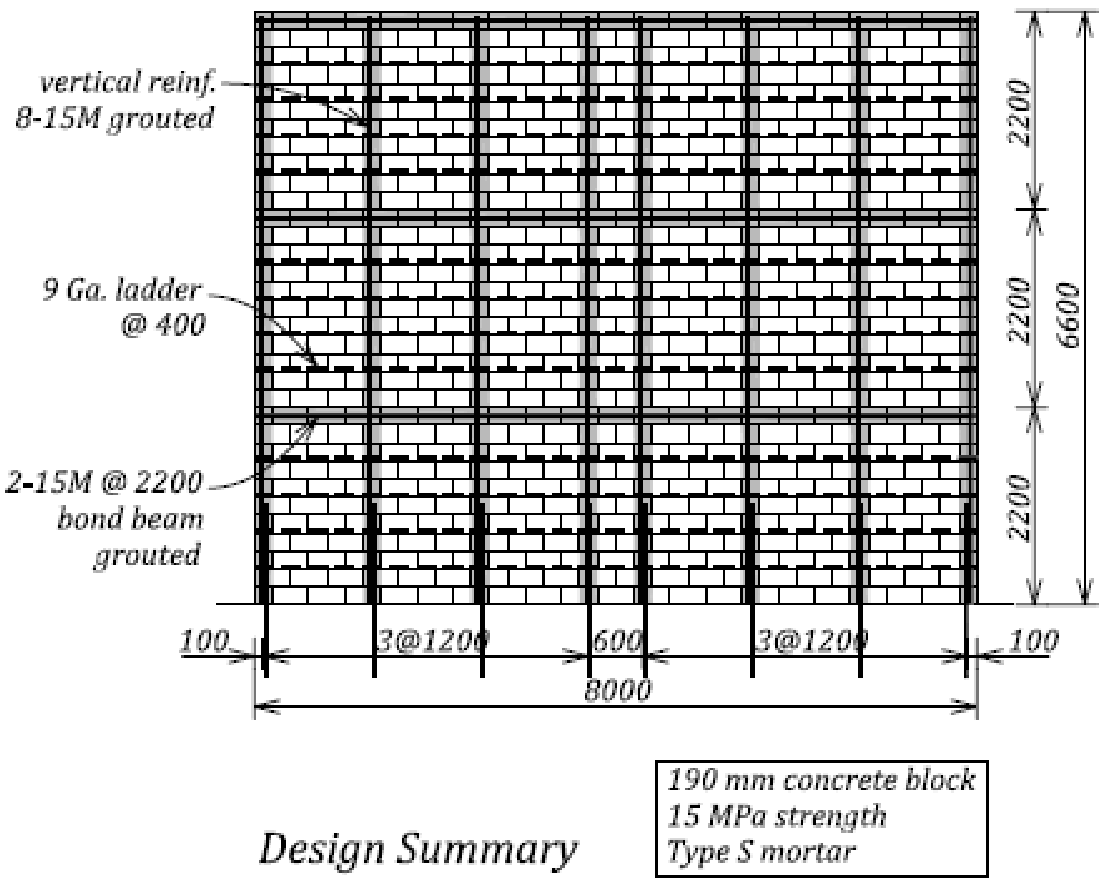
Since

$V_{nb} = 492 < V_r = 508$ kN

the capacity design criterion has been satisfied. <u>It should be noted that CSA S304.1-04 does not formally require that capacity design approach be applied to all categories of reinforced masonry walls – it is mandatory only for “ductile walls” (limited ductility and moderately ductile shear walls).</u> However, it is a good practice to consider the capacity design approach in designing all reinforced masonry walls in areas where seismic design is required by NBCC 2005 (note that this approach is followed in CSA A23.3 reinforced concrete design standard).

8. Design summary

The reinforcement arrangement for the wall under consideration is summarized below.



Design Summary

9. Discussion

It is important to consider all possible behaviour modes and identify the one that governs in this design. There are three shear forces:

a) $V_{nb} = 492$ kN shear force corresponding to flexural failure

b) $V_r = 508$ kN diagonal tension shear resistance

c) $V_r = 451$ kN sliding shear resistance

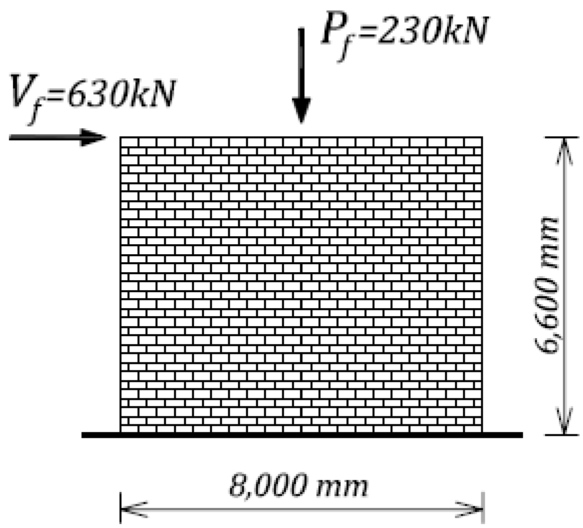
Since the sliding shear resistance value is the lowest, it can be concluded that the sliding shear mechanism is critical for this case, which is common for seismic design of squat shear walls.

As discussed at the beginning of this example, CSA S304.1 permits the minimum seismic reinforcement to be distributed in different ways. The solution presented above proposed that minimum reinforcement be placed in the vertical direction so that the capacity design criterion could be satisfied. Had the designer decided that more vertical reinforcement is required for meeting the flexural design requirements, (s)he could have used 10-15M bars instead of 8-15M bars (this would result in a 25% increase in the amount of vertical reinforcement). The corresponding amount of horizontal reinforcement could be reduced: 1-15M at 2200 mm spacing for bond beam reinforcement and 9 Ga. joint reinforcement at 400 mm spacing. This combination would meet the minimum seismic reinforcement requirements (250 mm^2/m vertical reinforcement and 146 mm^2/m horizontal reinforcement, with a total of 397 mm^2/m). For this arrangement of reinforcing the moment and shear resistance would be: $M_r = 3195$ kNm (a 16% increase compared to the previous value $M_r = 2762$ kNm) and $V_{nb} = 571$ kN shear force corresponding to flexural failure; however, $V_r = 392$ kN (diagonal tension shear resistance). Since $V_{nb} > V_r$ the capacity design criterion would not be met. As a result, this wall would be expected to fail in a shear (diagonal tension) mode characterized by a brittle failure, which is undesirable. Alternatively, the wall might fail in shear sliding mode, which is more desirable than the diagonal shear failure and often governs in low-rise reinforced masonry shear walls.

EXAMPLE 4b: Seismic design of a squat shear wall of conventional construction

Design a single-storey squat concrete block shear wall shown in the figure below according to NBCC 2005 and CSA S304.1 seismic requirements for conventional construction. The building site is located in Ottawa, ON on Site Class C soil, and the seismic hazard index $I_E F_a S_a(0.2)$ is 0.66. The wall is subjected to a total dead load of 230 kN (including the wall self-weight) and an in-plane seismic force of 630 kN. Consider the wall to be solid grouted. Neglect the out-of-plane effects in this design.

Use 200 mm hollow concrete blocks of 15 MPa unit strength and Type S mortar. Grade 400 steel reinforcing bars (yield strength f_y = 400 MPa) and cold-drawn galvanized wire (ASWG) joint reinforcement are used for this design.



Wall dimensions:
l_w =8000 mm length
h_w = 6600 mm height
t = 190 mm thickness

SOLUTION:

1. Material properties

Steel (both reinforcing bars and joint reinforcement):
ϕ_s = 0.85 f_y = 400 MPa
Masonry:
ϕ_m = 0.6
S304.1 Table 4, 15 MPa concrete blocks and Type S mortar:
f'_m = 7.5 MPa (assume solid grouted masonry)

2. Load analysis

The wall needs to be designed for the following load effects:

- P_f = 230 kN axial load
- V_f = 630 kN seismic shear force
- $M_f = V_f * h$ = 630*6.6 ≈ 4160 kNm overturning moment at the base of the wall

Note that, according to NBCC 2005 Table 4.1.3.2, load combination for the dead load and seismic effects is 1.0*D + 1.0*E.

3. Minimum CSA S304.1 seismic reinforcement (see Section 2.5.4.7 and Table 2-2)
Since $I_E F_a S_a(0.2)$= 0.66 > 0.35, minimum seismic reinforcement is required (S304.1 Cl.10.15.2.2). See Example 4a for a detailed calculation of the S304.1 minimum seismic reinforcement.

4. Design for the combined axial load and flexure
A design for the combined effects of axial load and flexure will be performed using two different procedures: i) by considering uniformly distributed vertical reinforcement, and ii) by considering concentrated and distributed reinforcement.

Distributed wall reinforcement (see Section C.1.1.2)
This procedure assumes uniformly distributed vertical reinforcement over the wall length. The total vertical reinforcement area can be estimated, and the estimate can be revised until the moment resistance value is sufficiently large. After a few trial estimates, the total area of vertical reinforcement was determined as

A_{vt} = 3200 mm^2 > 1016 mm^2 (minimum seismic reinforcement) - OK

Try 16-15M bars for vertical reinforcement.
The wall is subjected to axial load

P_f = 230 kN

The approximate moment resistance for the wall section is given by:

$\alpha_1 = 0.85 \qquad \beta_1 = 0.8$

$$\omega = \frac{\phi_s f_y A_{vt}}{\phi_m f'_m l_w t} = \frac{0.85*400*3200}{0.6*7.5*8000*190} = 0.159$$

$$\alpha = \frac{P_f}{\phi_m f'_m l_w t} = \frac{230*10^3}{0.6*7.5*8000*190} = 0.034$$

$$c = \frac{\omega + \alpha}{2\omega + \alpha_1 \beta_1} l_w = \frac{0.159 + 0.034}{2*0.159 + 0.85*0.8}(8000) = 1547 \text{ mm}$$

$$M_r = 0.5\phi_s f_y A_{vt} l_w \left(1 + \frac{P_f}{\phi_s f_y A_{vt}}\right)\left(1 - \frac{c}{l_w}\right) = 0.5*0.85*\frac{400}{1000}*3200*\frac{8000}{1000}\left(1 + \frac{230*10^3}{0.85*400*3200}\right)\left(1 - \frac{1544}{8000}\right)$$

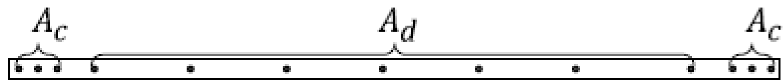
$M_r = 4253$ kNm > $M_f = 4160$ kNm OK

Distributed and concentrated wall reinforcement (see Section C.1.1.1)

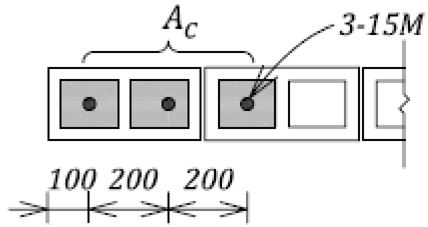
This procedure assumes the same total reinforcement area, but the concentrated reinforcement is provided at the wall ends, and the remaining reinforcement is distributed over the wall length.

A_{vt} = 3200 mm^2

Concentrated reinforcement area at each wall end (3-15M bars in total, 1-15M in last 3 cells):
A_c = 600 mm^2



Distributed reinforcement
A_d = 3200-2*600=2000 mm^2



Distance from the wall end to the centroid of concentrated reinforcement
$d' = 300$ mm

The compression zone depth a:

$$a = \frac{P_f + \phi_s f_y A_d}{0.85\phi_m f'_m t} = \frac{230*10^3 + 0.85*400*2000}{0.85*0.6*7.5*190}$$ = 1252 mm

The masonry compression resultant C_r:

$C_m = (0.85\phi_m f'_m)(t \cdot a) = (0.85*0.6*7.5)(190*1252) = 910$ kN

The factored moment resistance M_r will be determined by summing up the moments around the centroid of the wall section as follows (see equation (3) in Section C.1.1.1)

$$M_r = \left[C_m(l_w - a)/2 + 2(\phi_s f_y A_c)(l_w/2 - d')\right]*10^{-6}$$

$$= \left[910*10^3*(8000-1252)/2 + 2*(0.85*400*600)(8000/2-300)\right]*10^{-6}$$ $M_r = 4580$ kNm

The second procedure was used as a reference (to confirm the results of the first procedure). Both procedure give similar M_r values (4253 kNm and 4580 kNm by the first and second procedure respectively).

5. Find the diagonal tension shear resistance (see Section 2.3.2 and CSA S304.1 Cl.10.10.1).

Masonry shear resistance (V_m):

$b_w = 190$ mm overall wall thickness

$d_v \approx 0.8 l_w = 6400$ mm effective wall depth

$\gamma_g = 1.0$ solid grouted wall

$P_d = 0.9 P_f$ = 207 kN

$$v_m = 0.16(2 - \frac{M_f}{V_f d_v})\sqrt{f'_m}$$ = 0.44 MPa

$$\frac{M_f}{V_f d_v} = \frac{4160}{630*6.4}$$ = 1.03 ≈ 1.0

$V_m = \phi_m(v_m b_w d_v + 0.25 P_d)\gamma_g$ = 0.6(0.44*190*6400+0.25*207*10^3)*1.0 = 352 kN

Steel shear resistance V_s (2-15M bond beam reinforcement at 1200 mm spacing):

$$V_s = 0.6\phi_s A_v f_y \frac{d_v}{s} = 0.6 * 0.85 * \frac{400}{1000} * 400 * \frac{6400}{1200}$$ = 435 kN

Total shear resistance

$V_r = V_m + V_s = 352 + 435 = 787$ kN

Since

$V_r = 787$ kN > $V_f = 630$ kN   OK

This is a squat shear wall because $\frac{h_w}{l_w} = \frac{6600}{8000} = 0.825 \le 1.0$. Maximum shear allowed on the section is (S304.1 Cl.10.10.1.3)

$$\max V_r = 0.4\phi_m \sqrt{f'_m} b_w d_v \gamma_g (2 - \frac{h_w}{l_w}) = 939 \text{ kN}$$

Since

$V_r < \max V_r$ OK

Note that a solid grouted wall is required, that is, $\gamma_g = 1.0$. A partially grouted wall would have $\gamma_g = 0.5$, so its shear capacity would not be adequate for this design.

6. Sliding shear resistance (see Section 2.3.3)

The factored in-plane sliding shear resistance V_r is determined as follows.

μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

A_s = 3200 mm^2 total area of vertical wall reinforcement

$T_y = \phi_s A_s f_y$ = 0.85*3200*400 = 1088 kN

P_d = 207 kN

$P_2 = P_d + T_y$ = 207+1088 = 1295 kN

$V_r = \phi_m \mu P_2$ = 0.6*1.0*1295=777 kN

$V_r = 777$ kN > $V_f = 630$ kN   OK

7. Capacity design check (see Section 2.5.2)

At this point, both the moment resistance M_r and the diagonal shear resistance V_r for the wall section have been determined. It is a good seismic design practice to design structural members so that a ductile flexural failure takes place before a shear failure has been initiated, that is, to follow the capacity design approach discussed in Section 2.5.2. <u>Note that CSA S304.1-04 does not formally require that capacity design approach be applied to reinforced masonry walls of conventional construction – it is mandatory only for “ductile walls”.</u>

In this case, the factored moment resistance is equal to

$M_r = 4253$ kNm

The nominal moment resistance can be estimated as follows

$$M_n = \frac{M_r}{\phi_s} = \frac{4253}{0.85} = 5004 \text{ kNm}$$

The shear force at the top of the wall that would cause an overturning moment equal to M_n is

$$V_{nb} = \frac{M_n}{h_w} = \frac{5004}{6.6} = 758 \text{ kN}$$

To ensure that a flexural failure takes place before the diagonal shear failure, it is required that (see Figure 2-22)

$V_{nb} \le V_r$

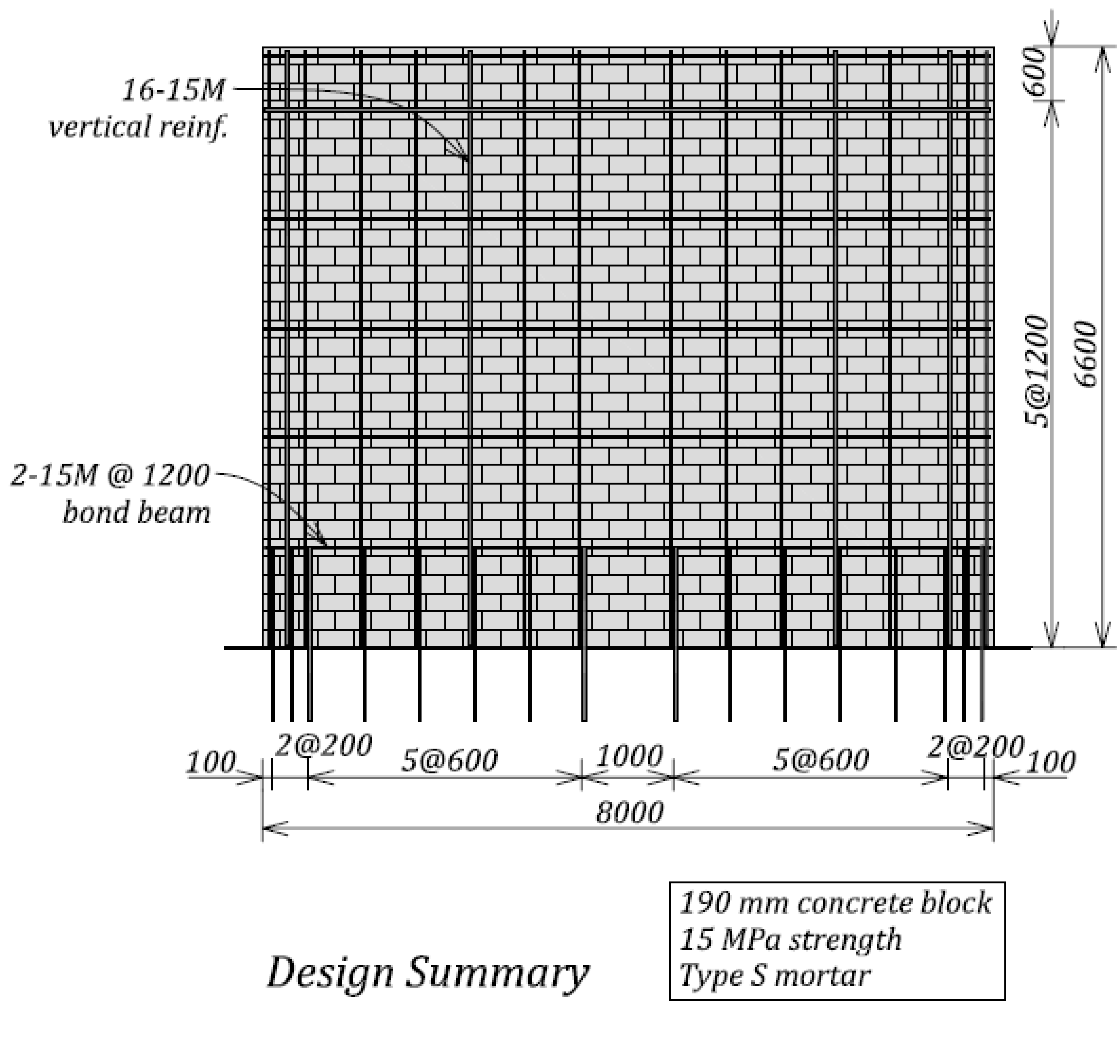
Since

$V_{nb} = 758 < V_r = 787$ kN

the capacity design criterion is satisfied (see discussion in Example 4a).

8. Design summary

The reinforcement arrangement for the wall under consideration is shown in the figure below. Note that the wall is solidly grouted. A bond beam (transfer beam) is provided atop the wall to ensure uniform shear transfer along the entire length (see Section 2.3.2.2).



Design Summary

10. Discussion

It is important to consider all possible behaviour modes and identify the one that governs in this design. There are three shear forces:

a) $V_{nb} = 758$ kN shear force corresponding to flexural failure

b) $V_r = 787$ kN diagonal tension shear resistance

c) $V_r = 777$ kN sliding shear resistance

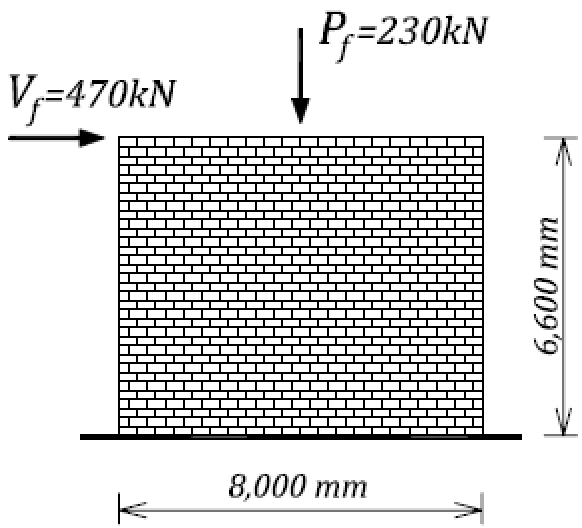
Since the shear force corresponding to flexural resistance is smallest of the three values, it can be concluded that the flexural failure mechanism is critical in this case, which is desirable for seismic design.

Note that CSA S304.1-04 Cl.10.2.8 prescribes the use of a reduced effective depth d for the flexural design of squat shear walls. This example deals with seismic design, and the wall reinforcement is expected to yield in tension, this provision was not followed since it would lead to a non-conservative design; instead, the actual effective depth was used for flexural design.

EXAMPLE 4c: Seismic design of a squat shear wall of moderate ductility

Design a single-storey squat concrete block shear wall shown on the figure below according to NBCC 2005 and CSA S304.1 seismic requirements for moderately ductile squat shear walls (note that the same shear wall was designed in Example 4b as a conventional construction). The building site is located in Ottawa, ON and the seismic hazard index $I_E F_a S_a(0.2)$ is 0.66. The wall is subjected to the total dead load of 230 kN (including the wall self-weight) and the in-plane seismic force of 470 kN; this reflects the higher R_d value of 2.0 that can be used for walls with moderate ductility. Consider the wall to be solid grouted. Neglect the out-of-plane effects in this design.

Use 200 mm hollow concrete blocks of 15 MPa unit strength and Type S mortar. Grade 400 steel reinforcing bars (yield strength f_y = 400 MPa) and cold-drawn galvanized wire (ASWG) joint reinforcement are used for this design.



Wall dimensions:
l_w =8000 mm length
h_w = 6600 mm height
t = 190 mm thickness

Note that the h/t ratio exceeds the S304.1 limit of 20 for moderately ductile squat shear walls (Cl.10.16.6.3).

SOLUTION:

Since

$$\frac{h_w}{l_w} = \frac{6600}{8000} = 0.825 \le 1.0$$

this is a squat shear wall (S304.1 Cl.4.6.6). The wall is to be designed as a moderately ductile squat shear wall, and NBCC 2005 Table 4.1.8.9 specifies the following R_d and R_o values (see Table 1-13):

R_d = 2.0 and R_o = 1.5

The seismic shear force of 470 kN for a wall with moderate ductility ($R_d = 2.0$) was obtained by prorating the force of 630 kN from Example 4b which corresponded to a shear wall with conventional construction ($R_d = 1.5$), as follows

$$V_f = 630 * \frac{1.5}{2.0} \approx 470 \text{ kN}$$

1. Material properties

Steel (both reinforcing bars and joint reinforcement):

ϕ_s = 0.85 f_y = 400 MPa

Masonry:
$\phi_m = 0.6$
From S304.1 Table 4, 15 MPa concrete blocks and Type S mortar:
$f'_m = 7.5$ MPa (assume solid grouted masonry)

2. Load analysis
The wall needs to be designed for the following load effects:

- P_f = 230 kN axial load
- V_f = 470 kN seismic shear force
- $M_f = V_f * h$ = 470*6.6 ≈ 3100 kNm overturning moment at the base of the wall

Note that, according to NBCC 2005 Table 4.1.3.2, the load combination for the dead load and seismic effects is 1.0*D + 1.0*E.

3. Minimum CSA S304.1 seismic reinforcement (see Section 2.5.4.7 and Table 2-2)
Since $I_E F_a S_a(0.2)$ = 0.66 > 0.35, minimum seismic reinforcement is required (Cl.10.15.2.2). See Example 4a for a detailed calculation of the S304.1 minimum seismic reinforcement.

4. Design for the combined axial load and flexure (see Section C.1.1.2).
A design for the combined effects of axial load and flexure will be performed by assuming uniformly distributed vertical reinforcement over the wall length. After a few trial estimates, the total area of vertical reinforcement was determined as
A_{vt} = 2200 mm^2 > 1016 mm^2 (minimum seismic reinforcement) - OK
and so 11-15M reinforcing bars can be used for vertical reinforcement in this design (total area of 2200 mm^2).

The wall is subjected to axial load P_f = 230 kN. Note that the load factor for the load combination with earthquake load is equal to 1.0.

The moment resistance for the wall section can be determined from the following equations (see Example 4b):
$\alpha_1 = 0.85 \quad \beta_1 = 0.8 \quad \omega = 0.109 \quad \alpha = 0.034 \quad c = 1273$ mm

$$M_r = 0.5\phi_s f_y A_{vt} l_w \left(1 + \frac{P_f}{\phi_s f_y A_{vt}}\right)\left(1 - \frac{c}{l_w}\right) = 0.5 * 0.85 * \frac{400}{1000} * 2200 * \frac{8000}{1000}\left(1 + \frac{230*10^3}{0.85*400*2200}\right)\left(1 - \frac{1273}{8000}\right)$$

$M_r \cong 3290$ kNm > $M_f = 3100$ kNm OK

5. Height/thickness ratio check (see Section 2.5.4.4)
CSA S304.1-04 prescribes the following height-to-thickness (h/t) limit for the compression zone in moderately ductile squat shear walls (Cl.10.16.6.3):
$h/(t+10) < 20$, unless it can be shown for lightly loaded walls that a more slender wall is satisfactory for out-of-plane stability.

For this example,
h = 6600 mm (unsupported wall height)

t = 190 mm actual wall thickness
So,
$h/(t+10) = 6600/(190+10) = 33 > 20$
The height-to-thickness ratio for this wall exceeds the CSA S304.1 limits by a significant margin. However, CSA S304.1 permits the height-to-thickness restrictions for moderately ductile squat shear walls to be relaxed, provided that the designer can show that the out-of-plane wall stability is satisfactory.

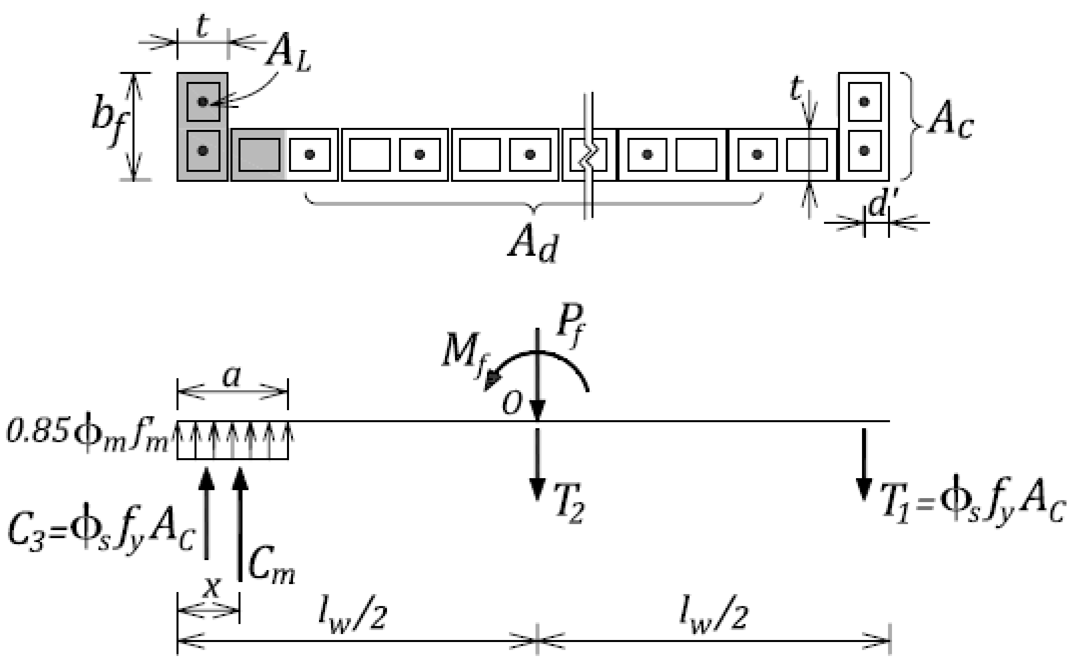
This is a lightly loaded wall in a single-storey building. The total dead load is 230 kN, which corresponds to the compressive stress of

$$f_c = \frac{P_f}{l_w t} = \frac{230*10^3}{8000*190} = 0.15 \text{ MPa}$$

This stress corresponds to only 2% of the masonry compressive strength f'_m which is equal to 7.5 MPa. In general, a compressive stress below 0.1 f'_m (equal to 0.75 MPa in this case) is considered to be very low.

The recommendations included in the commentary to Section 2.5.4.4 will be followed here. A possible solution involves the provision of flanges at the wall ends. The out-of-plane stability of the compression zone must be confirmed for this case.

Try an effective flange width $b_f = 390$ mm. The wall section and the internal force distribution is shown on the figure below.



This procedure assumes the same total reinforcement area A_{vt} as determined in step 4, but the concentrated reinforcement is provided at the wall ends, while the remaining reinforcement is distributed over the wall length.

A_{vt} = 2200 mm^2

Concentrated reinforcement area (2-15M bars at each wall end):

A_c = 400 mm^2

Distributed reinforcement area:

A_d =2200-2*400=1400 mm^2

Distance from the wall end to the centroid of concentrated reinforcement A_c :

$d' = 100$ mm

- Check the buckling resistance of the compression zone.

The area of the compression zone A_L :

$$A_L = \frac{P_f + \phi_s f_y A_d}{0.85\phi_m f'_m} = \frac{230*10^3 + 0.85*400*1400}{0.85*0.6*7.5} = 1.846*10^5 \text{ mm}^2$$

The depth of the compression zone a :

$$a = \frac{A_L - b_f * t + t^2}{t} = \frac{1.846*10^5 - (390*190) + 190^2}{190} = 772 \text{ mm}$$

The neutral axis depth:

$$c = \frac{a}{0.8} = 965 \text{ mm}$$

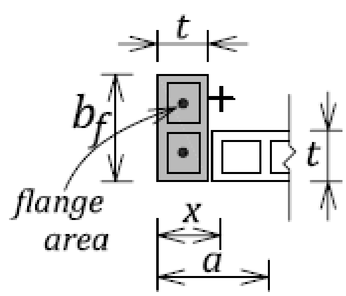
The centroid of the masonry compression zone:

$$x = \frac{t*\left(a^2/2\right) + (b_f - t)\left(t^2/2\right)}{A_L} = 326 \text{ mm}$$

In this case, the compression zone is L-shaped, however only the flange area will be considered for the buckling resistance check (see the shaded area shown on the figure below). This is a conservative approximation and it is considered to be appropriate for this purpose, since the gross moment of inertia is used.

Gross moment of inertia for the flange only:

$$I_{xg} = \frac{t*b_f^{\ 3}}{12} = \frac{190*390^3}{12} = 9.39*10^8 \text{ mm}^4$$



The buckling strength for the compression zone will be determined according to S304.1 Cl. 10.7.4.3, as follows:

$$P_{cr} = \frac{\pi^2 \phi_{er} E_m I}{(1 + 0.5\beta_d)(kH)^2} = 1017 \text{ kN}$$

where

$\phi_{er} = 0.75$

$k = 1.0$ pin-pin support conditions

$\beta_d = 0$ assume 100% seismic live load

$H = 6600$ mm wall height

$E_m = 850 f'_m = 6375$ MPa modulus of elasticity for masonry

- Find the resultant compression force (including the concrete and steel component).

$$P_{fb} = C_m + \phi_s f_y A_c = 706*10^3 + 0.85*400*400 = 842 \text{ kN}$$

where

$$C_m = (0.85\phi_m f'_m)A_L = (0.85*0.6*7.5)(1.846*10^5) = 706 \text{ kN}$$

- Confirm that the out-of-plane buckling resistance is adequate.

Since

$P_{fb} = 842$ kN < $P_{cr} = 1017$ kN

it can be concluded that the out-of-plane buckling resistance is adequate and so the flanged section can be used for this design. This is in compliance with S304.1 Cl.10.16.6.3, despite the fact that the h/t ratio for this wall is 33, which exceeds the CSA S304.1-prescribed limit of 20.

4a. Design the flanged section for the combined axial load and flexure – consider distributed and concentrated wall reinforcement (see Section C.1.1.1).

The key design parameters for this calculation were determined in step 5 above. The factored moment resistance M_r will be determined by summing up the moments around the centroid of the wall section as follows

$$M_r = C_m\left(l_w/2 - x\right) + 2\left(\phi_s f_y A_c\right)\left(l_w/2 - d'\right) = 706*10^3*(8000/2 - 326) + 2*\left(0.85*400*400\right)*(8000/2 - 100)$$

$M_r = 3655*10^6\ Nmm = 3655$ kNm

Since

$M_r = 3655$ kNm > $M_f = 3100$ kNm OK

6. The diagonal tension shear resistance (see Section 2.3.2 and CSA S304.1 Cl.10.10.1)

Masonry shear resistance (V_m):

$b_w = 190$ mm overall wall thickness

$d_v \approx 0.8 l_w = 6400$ mm effective wall depth

$\gamma_g = 1.0$ solid grouted wall

$P_d = 0.9 P_f$ = 207 kN

$v_m = 0.16(2 - \frac{M_f}{V_f d_v})\sqrt{f'_m}$ = 0.44 MPa

$\frac{M_f}{V_f d_v} = \frac{3100}{470*6.4}$ = 1.03 ≈ 1.0

$V_m = \phi_m(v_m b_w d_v + 0.25 P_d)\gamma_g$ = 0.6(0.44*190*6400+0.25*207*10^3)*1.0 = 352 kN

Steel shear resistance V_s:

Assume 2-15M bond beam reinforcement at 1200 mm spacing, so

$A_v = 400$ mm^2

$s = 1200$ mm

Horizontal reinforcement area per metre:

$A_h{}' = \frac{A_v}{s}*1000 = \frac{400}{1200}*1000 = 333$ mm^2/m

$V_s = 0.6\phi_s A_v f_y \frac{d_v}{s} = 0.6*0.85*\frac{400}{1000}*400*\frac{6400}{1200}$ = 435 kN

Total diagonal shear resistance

$V_r = V_m + V_s = 352 + 435 = 787$ kN

$V_r = 787$ kN > $V_f = 470$ kN OK

Maximum shear allowed on the section is (S304.1 Cl.10.10.1.3)

$$\max V_r = 0.4\phi_m \sqrt{f'_m} b_w d_v \gamma_g (2 - \frac{h_w}{l_w}) = 939 \text{ kN}$$

Since

$V_r < \max V_r$ OK

Note that CSA S304.1 Cl.10.16.6.2 requires that the method by which the shear force is applied to the wall shall be capable of applying shear force uniformly over the wall length. This can be achieved by providing a continuous bond beam at the top of the wall, as discussed in Section 2.3.2.2 (see Figure 2-16).

7. Sliding shear resistance (see Section 2.3.3)

The factored in-plane sliding shear resistance V_r is determined as follows.

μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

A_s = 2200 mm^2 total area of vertical wall reinforcement

$T_y = \phi_s A_s f_y$ = 0.85*2200*400 = 748 kN

P_d = 207 kN

$P_2 = P_d + T_y$ = 207+748 = 955 kN

$V_r = \phi_m \mu P_2$ = 0.6*1.0*955 = 573 kN

$V_r = 573$ kN > $V_f = 470$ kN OK

Note that $V_r = 573$ kN < $V_{nb} = 787$ kN (this indicates that the sliding shear resistance governs over the diagonal tension shear resistance).

8. Reinforcement requirements for moderately ductile squat shear walls (see Section 2.5.4.8)

CSA S304.1-04 Cl.10.16.6.6 introduced the following new requirements for the amount of reinforcement in moderately ductile squat shear walls:

<u>Vertical reinforcement ratio ρ_v</u>

Actual vertical reinforcement ratio ρ_{vflex} based on the flexural design requirements (see step 4):

$$\rho_{vflex} = \frac{A_{vt}}{l_w * t} = \frac{2200}{8000*190} = 1.447*10^{-3}$$

Minimum $\rho_{v\min}$ value set by CSA S304.1 Cl.10.16.6.6.1:

$$\rho_{v\min} \geq \frac{V_f - P_f}{\phi_s \cdot b_w \cdot l_w \cdot f_y} = \frac{470*10^3 - 230*10^3}{0.85*190*8000*400} = 0.464*10^{-3}$$

Since

ρ_{vflex} = 1.447*10^{-3} > $\rho_{v\min}$ = 0.464*10^{-3}

Therefore, the amount of vertical reinforcement determined based on the flexural design requirements (11-15M) is OK.

Horizontal reinforcement ratio ρ_h

ρ_h should be greater of

a) the minimum value set by CSA S304.1 Cl.10.16.6.6.2:

$$\phi_s \rho_{h\min} \geq \phi_s \rho_v + \frac{P_f}{b_w l_w f_y} = 0.85 * 4.644 * 10^{-4} + \frac{230 * 10^3}{190 * 8000 * 400} = 0.773*10^{-3}$$

(note that the vertical reinforcement ratio used in this relation is the one that governed above, that is, $\rho_v = \rho_{v\min}$ = 4.644*10^-4)

and

b) the value determined in accordance with Cl.10.10 based on the shear resistance requirements

$$\phi_s \rho_{hshear} = \frac{\phi_s A_h}{b_w * h_w} = \frac{0.85 * 2131}{190 * 6600} = 1.44 * 10^{-3}$$

where A_h is the total area of horizontal reinforcement along the wall height, that is,

$A_h = A_h' * d_v$ = 333*6.4=2131 mm²

$A_h' = 333$ mm²/m (see step 6)

In this case,

$\phi_s \rho_{h\min}$ =0.773*10^-3 < $\phi_s \rho_{hshear}$ =1.44*10^-3

This indicates that the CSA S304.1 shear resistance requirement governs. The amount of horizontal reinforcement (2-15M bond beam reinforcement bar at 1200 mm spacing) is adequate.

9. Capacity design check (see Section 2.5.2)

At this point, both the moment resistance M_r and the diagonal shear resistance V_r for the wall section have been determined. S304.1 Cl.10.16.3.3 requires that ductile reinforced masonry shear walls be designed so that flexural failure takes place before shear failure has been initiated, that is, to follow the capacity design approach (see Section 2.5.2 for more details).
In this case, the factored moment resistance is equal to

$M_r = 3655$ kNm

The nominal moment resistance can be estimated as follows

$$M_n = \frac{M_r}{\phi_s} = \frac{3655}{0.85} = 4300 \text{ kNm}$$

The shear force at the top of the wall that would cause an overturning moment equal to M_n is

$$V_{nb} = \frac{M_n}{h_w} = \frac{4300}{6.6} = 652 \text{ kN}$$

In order to ensure that flexural failure takes place before the diagonal shear failure, it is required that (see Figure 2-22)

$V_{nb} \leq V_r$

Since

$V_{nb} = 652 < V_r = 787$ kN

the capacity design criterion is satisfied (see discussion in Example 4a).

10. Shear resistance at the web-to-flange interface (see Section C.2 and Cl.7.11.4).
The factored shear stress at the web-to-flange interface is equal to the larger of horizontal and vertical shear stress, as shown below.
Horizontal shear:

$$v_f = \frac{V_f}{t_e l_w} = \frac{470*10^3}{190*8000} = 0.31 \text{ MPa}$$

where t_e = 190 mm (effective wall thickness)

Vertical shear (caused by the resultant compression force P_{fb} calculated in Step 5):

$$v_f = \frac{P_{fb}}{b_w * h_w} = \frac{842*10^3}{190*6600} = 0.67 \text{ MPa} \qquad \text{governs}$$

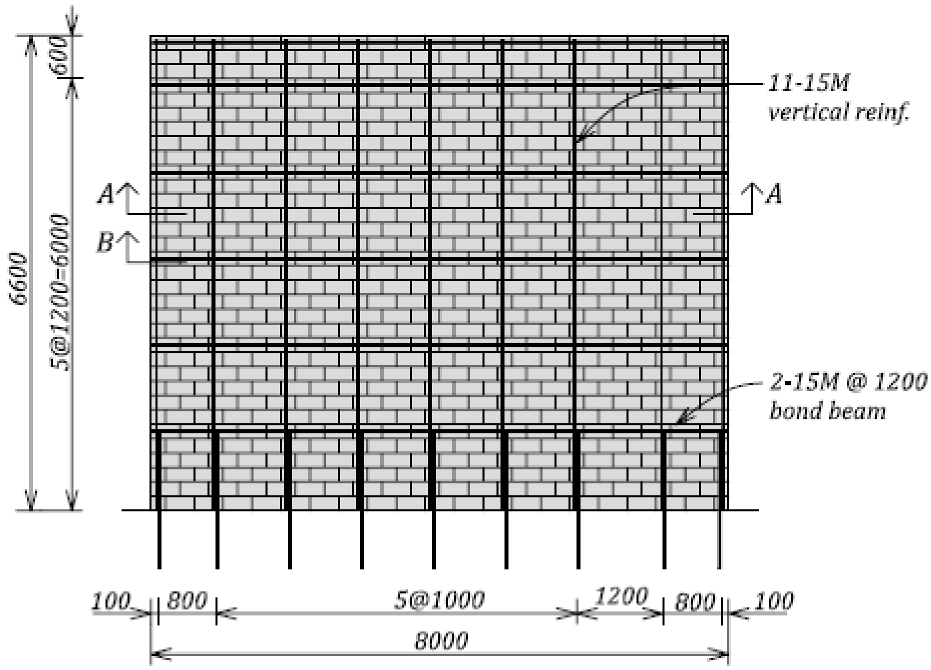
Masonry shear resistance:
$v_m = 0.44$ MPa (see step 6)
Since
$v_f = 0.67$ MPa > $\phi_m v_m = 0.26$ MPa
shear reinforcement at the web-to-flange interface is required. Since the horizontal reinforcement consists of 2-15M bars @ 1200 mm spacing, both bars can be extended into the flange (90° hook), and so

$$v_s = \frac{\phi_s A_b f_y}{s \cdot t_e} = \frac{0.85*2*200*400}{1200*190} = 0.60 \text{ MPa}$$

The total shear resistance
$v_r = \phi_m v_m + v_s = 0.26 + 0.60 = 0.86$ MPa
Since
$v_f = 0.67$ MPa < $v_r = 0.86$ MPa
the shear resistance at the web-to-flange interface is satisfactory.

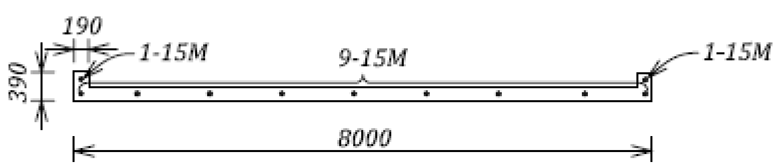
12. Design summary

The reinforcement arrangement for the wall under consideration is shown in the figure below. Note that the wall is solid grouted.

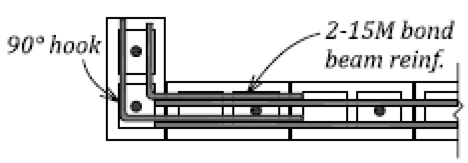


Design Summary

190 mm concrete block
15 MPa strength
Type S mortar



Section A-A



Section B-B

13. Discussion

It is important to consider all possible behaviour modes and identify the one that governs in this design. There are three shear forces:

a) $V_{nb} = 652$ kN shear force corresponding to flexural failure

b) $V_r = 787$ kN diagonal tension shear resistance

c) $V_r = 573$ kN sliding shear resistance

Since the sliding shear resistance value is smallest, it can be concluded that the sliding shear mechanism is critical in this case, which is common for seismic design of squat shear walls. Sliding shear resistance can be increased by roughening the wall-to-foundation interface (in which case the frictional coefficient can be increased to μ = 1.4) or by providing shear keys. Alternatively, additional dowels could be provided at the base of the wall, however this would result in an increase in the moment resistance. The designer would need to ensure that the capacity design criterion discussed in step 9 is satisfied.

Note that CSA S304.1-04 Cl.10.2.8 prescribes the use of reduced effective depth d for flexural design of squat shear walls. Since this example deals with seismic design and essentially all the wall reinforcement is expected to yield in tension, this provision was not used as it is expected to result in additional vertical reinforcement, which would increase the moment capacity and possibly lead to a more brittle diagonal shear failure.

Note that the S304.1 ductility check is not prescribed for moderately ductile squat shear walls (this requirement applies to the narrower flexural shear walls of moderate ductility per Cl.10.16.5.2.3).

This example shows that an addition of flanges can be effective in preventing the out-of-plane buckling of moderately ductile squat shear walls. This is in compliance with S304.1 Cl.10.16.6.3, despite the fact that the h/t ratio for this wall is 33, which exceeds the CSA S304.1-prescribed limit of 20.

The last two examples provide an opportunity for comparing the total amount of vertical reinforcement required for a squat shear wall of conventional construction (Example 4b) and a moderately ductile squat shear wall (this example). It is noted that the moderately ductile wall has less vertical reinforcement (11-15M bars) than a similar wall of conventional construction (16-15M bars); this reduction amounts to approximately 30%.

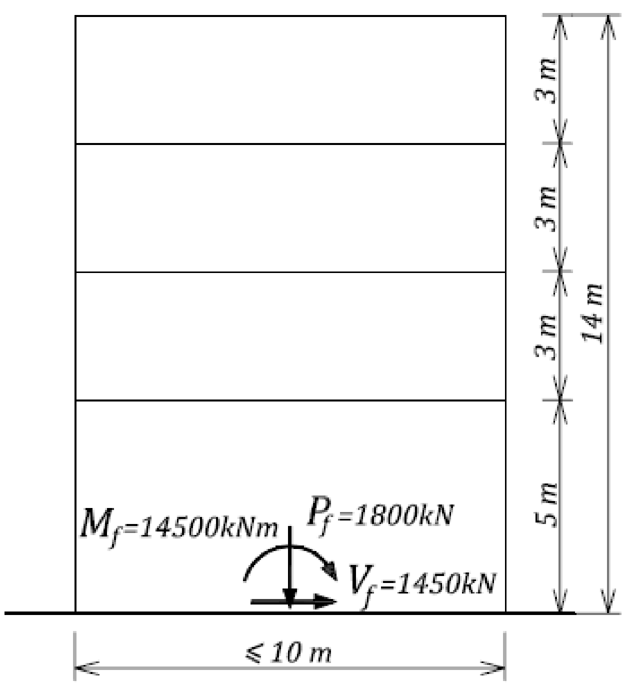
<u>EXAMPLE 5a</u>: Seismic design of a flexural shear wall of limited ductility

Perform the seismic design of a shear wall X_1, which is a part of the building discussed in Example 2. The wall is four storeys high, with the total height of 14 m, and due to its height must be designed either as a "limited ductility" or a "moderate ductility" shear wall per NBCC 2005 Table 4.1.8.9 (same as Table 1-13 in Chapter 1 of this document).

The section at the base of the wall is subjected to the total dead load of 1800 kN, the in-plane seismic shear force of 1450 kN, and the overturning moment of 14500 kNm. Select the wall dimensions (length and thickness) and the reinforcement such that the CSA S304.1 Cl.10.16.4 seismic design requirements for limited ductility shear walls are satisfied. Due to architectural constraints, the wall length should not exceed 10 m, and a rectangular (unflanged) wall section should be used.

Use hollow concrete blocks of 20 MPa unit strength and Type S mortar. Consider the wall as solid grouted. Grade 400 steel reinforcement (yield strength f_y = 400 MPa) is used for this design.

Note: the wall dead load was calculated based on the tributary area (3.4 m by 13.4 m) at each floor level (see a typical floor plan shown in Example 2), plus the wall self-weight.



<u>SOLUTION:</u>

1. Material properties

Steel (both reinforcing bars and joint reinforcement):

ϕ_s = 0.85 f_y = 400 MPa

Masonry:

ϕ_m = 0.6

S304.1 Table 4, 20 MPa concrete blocks and Type S mortar:

f'_m = 10.0 MPa (assume solid grouted masonry)

2. Load analysis

The section at the base of the wall needs to be designed for the following load effects:

- P_f = 1800 kN axial load
- V_f = 1450 kN seismic shear force
- M_f = 14500 kNm overturning moment

According to S304.1 Cl.4.6.4, this is a flexural shear wall because h_w = 14000 mm height and l_w = 10000 mm length, and

$$\frac{h_w}{l_w} \geq \frac{14000}{10000} \geq 1.4 > 1.0$$

and so the CSA S304.1 seismic design requirements for limited ductility (flexural) shear walls should be followed.

3. Determine the required wall thickness based on the S304.1 height-to-thickness requirements (Cl.10.16.4.1.2, see Section 2.5.4.4)

CSA S304.1-04 prescribes the following height-to-thickness (h/t) limit for the compression zone in limited ductility shear walls:

$$h/(t+10) < 18$$

For this example,

h = 5000 mm (the largest unsupported wall height)

So,

$t \geq h/18 - 10 = 268$ mm

Therefore, in this case the only possible wall thickness is

$t = 290$ mm

Alternatively, the designer may wish to consider a flanged wall section with smaller thickness. This is possible, except that (s)he would need to prove that the out-of-plane wall stability is not a concern (see Example 5b).

4. Determine the wall length based on the shear design requirements.

Designers may be requested to determine the wall dimensions (length and thickness) based on the design loads. In this case, the thickness is governed by the height-to-thickness ratio requirements, and the length can be determined from the maximum shear resistance for the wall section. The shear resistance for flexural walls cannot exceed the following limit (S304.1 Cl.10.10.1.1):

$$V_r \leq \max V_r = 0.4\phi_m \sqrt{f'_m} b_w d_v \gamma_g$$

$\gamma_g = 1.0$ solid grouted wall (required for plastic hinge zone)

$b_w = 290$ mm overall wall thickness

$d_v \approx 0.8 l_w$ effective wall depth

Set

$V_r = V_f = 1450$ kN

and so

$$l_w > \frac{V_f}{0.4\phi_m\sqrt{f'_m}b_w(0.8)\gamma_g} = \frac{1450*10^3}{0.4*0.6*\sqrt{10}*290*0.8*1.0} = 8235 \text{ mm}$$

Therefore, based on the shear design requirements the designer could select the wall length of 8.4 m. However, a preliminary capacity design check indicated that a minimum wall length of nearly 10 m was required, thus try

$l_w = 10000$ mm

which gives

$\max V_r = 1760$ kN

5. Minimum CSA S304.1 seismic reinforcement requirements (see Table 2-2)

Since $I_E F_a S_a(0.2)$ = 0.95 > 0.35, it is required to provide minimum seismic reinforcement (S304.1 Cl.10.15.2.2). See Example 4a for a detailed discussion on the S304.1 minimum seismic reinforcement requirements.

6. Design for the combined axial load and flexure (see Section C.1.1.2).

Design for the combined effects of axial load and flexure will be performed by assuming uniformly distributed vertical reinforcement over the wall length. After a few trial estimates, the total area of vertical reinforcement was determined as follows

A_{vt} = 6000 mm^2

20-20M reinforcing bars can be used for vertical reinforcement in this design, and the average spacing is equal to

$$s \le \frac{10000-200}{19} = 516 \text{ mm}$$

Since the amount of vertical reinforcement is significant, it is required to check the maximum reinforcement area per S304.1 Cl.10.15.3 (see Table 2-2).

Since $s = 516mm$ < $4t = 4*290 = 1160mm$

$A_{s\max} = 0.02A_g = 0.02(290*10^3) = 5800$ mm^2/m

This corresponds to the total reinforcement area of approximately 58000 mm^2 for a 10 m long wall; this is significantly larger than the estimated area of vertical reinforcement.

The wall is subjected to axial load P_f = 1800 kN. The moment resistance for the wall section can be determined from the following equations (see Section C.1.1.2):

$\alpha_1 = 0.85 \quad \beta_1 = 0.8 \quad \omega = 0.12 \quad \alpha = 0.1 \quad c \approx 2400$ mm

$$M_r = 0.5\phi_s f_y A_{vt} l_w\left(1+\frac{P_f}{\phi_s f_y A_{vt}}\right)\left(1-\frac{c}{l_w}\right) = 0.5*0.85*\frac{400}{1000}*6000*\frac{10000}{1000}\left(1+\frac{1800*10^3}{0.85*400*6000}\right)\left(1-\frac{2400}{10000}\right)$$

$M_r = 14600$ kNm > $M_f = 14500$ kNm OK

7. Perform the CSA S304.1 ductility check (see Section 2.5.4.3).

To satisfy the CSA S304.1 ductility requirements for limited ductility shear walls (Cl.10.16.4.1.4), neutral axis depth ratio (c/l_w) should be less than the following limit:

$c/l_w < 0.2$ when $h_w/l_w < 6$

In this case, the neutral axis depth

c = 2400 mm

and so

$c/l_w = 2400/10000 = 0.24 > 0.2$

Therefore, the CSA S304.1 ductility requirement is not satisfied. However, Cl.10.16.4.1.4 also states that the maximum compressive strain in masonry in the plastic hinge zone shall be shown to not exceed 0.0025 at the desired ductility level.

At this point, the designer can use one of the following two alternative approaches to check whether the ductility is adequate per CSA S304.1 Cl.10.16.4.1.4:

1) Find the required wall length such that the c/l_w limit prescribed in the CSA S304.1 ductility criteria is satisfied.

The wall length can be estimated from Table D-2, which provides c/l_w ratios for different input parameters (α and ω). By inspection, it can be concluded that $c/l_w < 0.2$ when $\alpha \le 0.1$. Let us try to estimate the wall length based on this criterion.

Since

$$\alpha = \frac{1667 * P_f}{f'_m \, l_w t}$$

set

$$\alpha = 0.09 < 0.1$$

and so

$$l_w = \frac{1667 * P_f}{f'_m * \alpha * t} = \frac{1667 * 1800}{10.0 * 0.09 * 290} = 11496 \text{ mm}$$

Therefore, we can select an increased wall length $l_w = 11600$ mm.

2) Calculate the masonry strain in the extreme compression fibre based on the given design loads, and prove that its value is less than 0.0025.

This check will be performed based on the procedure explained in Section B.2 (see Figure B-5). The maximum displacement in the wall X_1 at the roof level was determined in Example 2 (step 8), that is,

$$\Delta_{max} = 46mm$$

Note that the above value includes only translational displacement component. Since the wall X_1 is located close to the centre of resistance, the torsional displacement component is not significant. In the case of wall X_1, torsional displacement has a different direction from the translational displacement and (if included) the total displacement would be less than the translational one.

The maximum displacement Δ_{max} is equal to the sum of elastic displacement at the onset of steel yielding Δ_y and the plastic (post-yield) displacement Δ_p, that is,

$$\Delta_{max} = \Delta_y + \Delta_p$$

The yield curvature (corresponding to the onset of yielding in steel reinforcement) can be estimated as follows

$\varphi_y = \frac{0.0035}{l_w} = \frac{0.0035}{10000} = 3.5 * 10^{-7}$ The elastic displacement at the effective height can be estimated as

$$\Delta_{ye} = \frac{\varphi_y h_e^2}{3} = \frac{(3.5 * 10^{-7})(10000)^2}{3} = 11.7mm$$

The elastic displacement at the top of the wall is equal to (see the discussion in Example 2, step 8)

$\Delta_y = 1.5 * \Delta_{ye} = 1.5 * 11.7 = 17.5mm$

So, the plastic displacement can be determined as

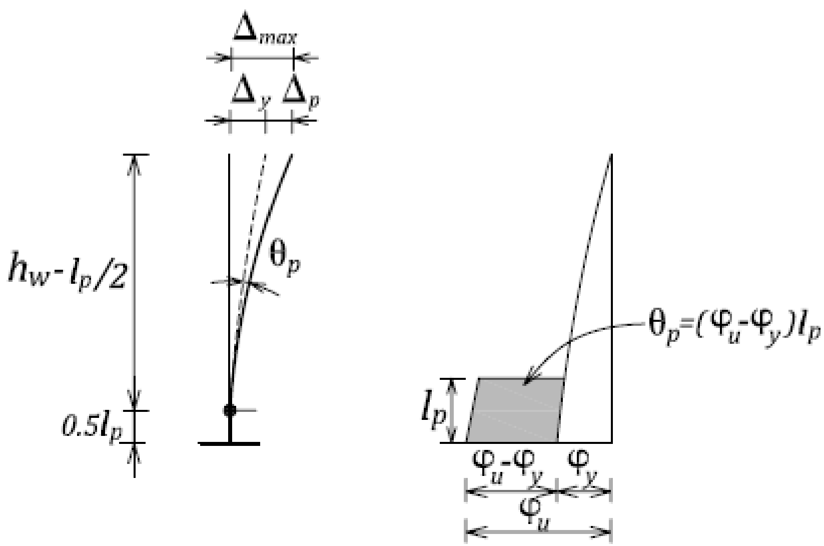
$\Delta_p = \Delta_{max} - \Delta_y = 46 - 17.5 = 28.5mm$

The plastic rotation θ_p can be found from the plastic displacement at the top and assuming that the plastic hinge has developed at the base is equal to (see the figure below)

$$\theta_p = \frac{\Delta_p}{h_w - \frac{l_p}{2}} = \frac{28.5}{14000 - \frac{5000}{2}} = 2.48 * 10^{-3} \text{ rad}$$

where the plastic hinge length to be used for ductility calculations has been estimated as

$l_p = 0.5 l_w = 0.5 * 10000 = 5000mm$



The maximum curvature can be determined from the following relationship between the rotation and the curvature:

$\theta_p = (\varphi_u - \varphi_y) * l_p$

and so

$$\varphi_u - \varphi_y = \frac{\theta_p}{l_p} = \frac{2.48 * 10^{-3}}{5000} = 4.96 * 10^{-7}$$

The ultimate curvature can then be determined as

$\varphi_u = 4.96 * 10^{-7} + 3.5 * 10^{-7} = 8.46 * 10^{-7}$

The maximum compressive strain in masonry can be determined from the following equation

$$\varphi_u = \frac{\varepsilon_m}{c}$$

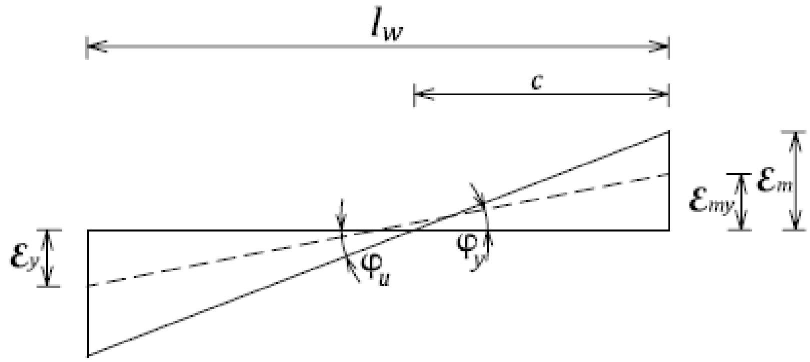
where

$c = 2400$ mm neutral axis depth (see step 6) and so

$\varepsilon_m = \varphi_u * c = (8.46 * 10^{-7})(2400) \approx 0.002$

It should be noted that this procedure uses an assumption that the neutral axis depth c has the same value at the onset of yielding (corresponding to strain ε_y) and at the ultimate (corresponding to strain ε_m); this is not true, however it does not significantly influence the accuracy of numerical results.

Since $\varepsilon_m = 0.002 < 0.0025$ it can be concluded that the wall satisfies the CSA S304.1 ductility requirements and that it is not necessary to increase its length. Therefore, the wall length $l_w = 10000$ mm will be used in the next steps. It should be noted that a larger wall length obtained from the first approach ($l_w = 11600$ mm) would have resulted in a reduced amount of vertical and horizontal reinforcement for the same flexural and shear design requirements, and would be a viable design solution had the wall length not been limited to 10 m due to architectural constraints.



8. The diagonal tension shear resistance and capacity design check (see Section 2.3.2 and CSA S304.1 Cl.10.10.1)

Masonry shear resistance (V_m):

$b_w = 290$ mm overall wall thickness

$d_v \approx 0.8 l_w = 8000$ mm effective wall depth

$\gamma_g = 1.0$ solid grouted wall

$P_d = 0.9 P_f$ = 1620 kN

$v_m = 0.16(2 - \frac{M_f}{V_f d_v})\sqrt{f'_m}$ = 0.51 MPa

Since

$\frac{M_f}{V_f d_v} = \frac{14500}{1450 * 8.0}$ = 1.25 > 1.0

use $\frac{M_f}{V_f d_v} = 1.0$

$V_m = \phi_m (v_m b_w d_v + 0.25 P_d)\gamma_g$ = 0.6(0.51*290*8000+0.25*1620*10^3)*1.0 = 953 kN

S304.1 Cl.10.16.3.3 requires that ductile reinforced masonry shear walls be designed according to the capacity design approach (see Section 2.5.2 for more details). According to that approach, the shear capacity should exceed the shear corresponding to the nominal moment resistance (see Figure 2-22), as follows

$M_n = \frac{M_r}{\phi_s} = \frac{14600}{0.85} = 17176$ kNm

where

$M_r = 14600$ kNm the factored moment resistance (see Step 6).

Shear force acts at the effective height h_e, that is, distance from the base of the wall to the resultant of all seismic forces acting at floor levels. h_e can be determined as follows

$$h_e = \frac{M_f}{V_f} = 10.0 \text{ m}$$

The shear force V_{nb} that would cause the overturning moment equal to M_n can be found as follows

$$V_{nb} = \frac{M_n}{h_e} = \frac{17176}{10.0} = 1718 \text{ kN}$$

This is less than the maximum shear allowed on the section (S304.1 Cl.10.10.1.1)

$$\max V_r = 0.4\phi_m \sqrt{f'_m} b_w d_v \gamma_g = 1760 \text{ kN} \quad \text{OK}$$

Thus the required steel shear resistance is

$$V_s = V_r - V_m = 1718 - 953 = 765 \text{ kN}$$

The required amount of reinforcement can be found from the following equation

$$\frac{A_v}{s} = \frac{V_s}{0.6\phi_s f_y d_v} = \frac{765*10^3}{0.6*0.85*400*8000} = 0.47$$

Try 2-15M bond beam reinforcing bars at 800 mm spacing ($A_v = 400$ mm^2 and $s = 800$ mm):

$$\frac{A_v}{s} = \frac{400}{800} = 0.5 > 0.47 \quad \text{OK}$$

Steel shear resistance V_s:

$$V_s = 0.6\phi_s A_v f_y \frac{d_v}{s} = 0.6*0.85*\frac{400}{1000}*400*\frac{8000}{800} = 816 \text{ kN}$$

Total diagonal shear resistance:

$$V_r = V_m + V_s = 953 + 816 = 1769 \text{ kN}$$

Since

$$V_r = 1769 \text{ kN} > V_f = 1450 \text{ kN} \quad \text{OK}$$

In conclusion, both the shear design requirements and the capacity design requirements have been satisfied.

9. Sliding shear resistance (see Section 2.3.3)

The factored in-plane sliding shear resistance V_r is determined as follows:

μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

A_s = 6000 mm^2 total area of vertical wall reinforcement

$T_y = \phi_s A_s f_y$ = 0.85*6000*400 = 2040 kN

$P_d = 0.9 P_f$ = 1620 kN

$P_2 = P_d + T_y$ = 1620+2040 = 3660 kN

$V_r = \phi_m \mu P_2$ = 0.6*1.0*3660 = 2196 kN

$V_r = 2196$ kN > $V_f = 1450$ kN OK

Also,

$V_r = 2196$ kN > $V_{nb} = 1718$ kN (capacity design check)

10. CSA S304.1 seismic detailing requirements for limited ductility walls – plastic hinge region

According to Cl.10.16.4.1.1, the required height of the plastic hinge region for limited ductility shear walls (for which special detailing is required) must be greater than (see Table 2-4)

$l_p = l_w / 2 = 10.0/2 = 5.0$ m

or

$l_p = h_w / 6 = 14.0 / 6 = 2.3$ m

(note that h_w denotes the total wall height)

Thus,

$l_p = 5.0$ m governs

Reinforcement detailing requirements for the plastic hinge region of limited ductility shear walls are:

1. ***The wall in the plastic hinge region must be solid grouted (Cl.10.16.4.1.3, see Table 2-4).***

2. ***Horizontal reinforcement requirements (see Figure 2-31)***

a) Reinforcement spacing should not exceed the following limits (Cl.10.16.4.3.3), see Table 2-2:

$s \le 1200$ mm or

$s \le l_w/2 = 10000/2 = 5000$ m

Since the lesser value governs, the maximum permitted spacing is

$s \le 1200$ mm

According to the design (see step 8), the horizontal reinforcement consists of 2-15M bars at 800 mm spacing - OK

b) Detailing requirements (Cl.10.16.4.3.3), see Table 2-3:

Horizontal reinforcement shall not be lapped within

600 mm or

c = 2400 mm (the neutral axis depth)

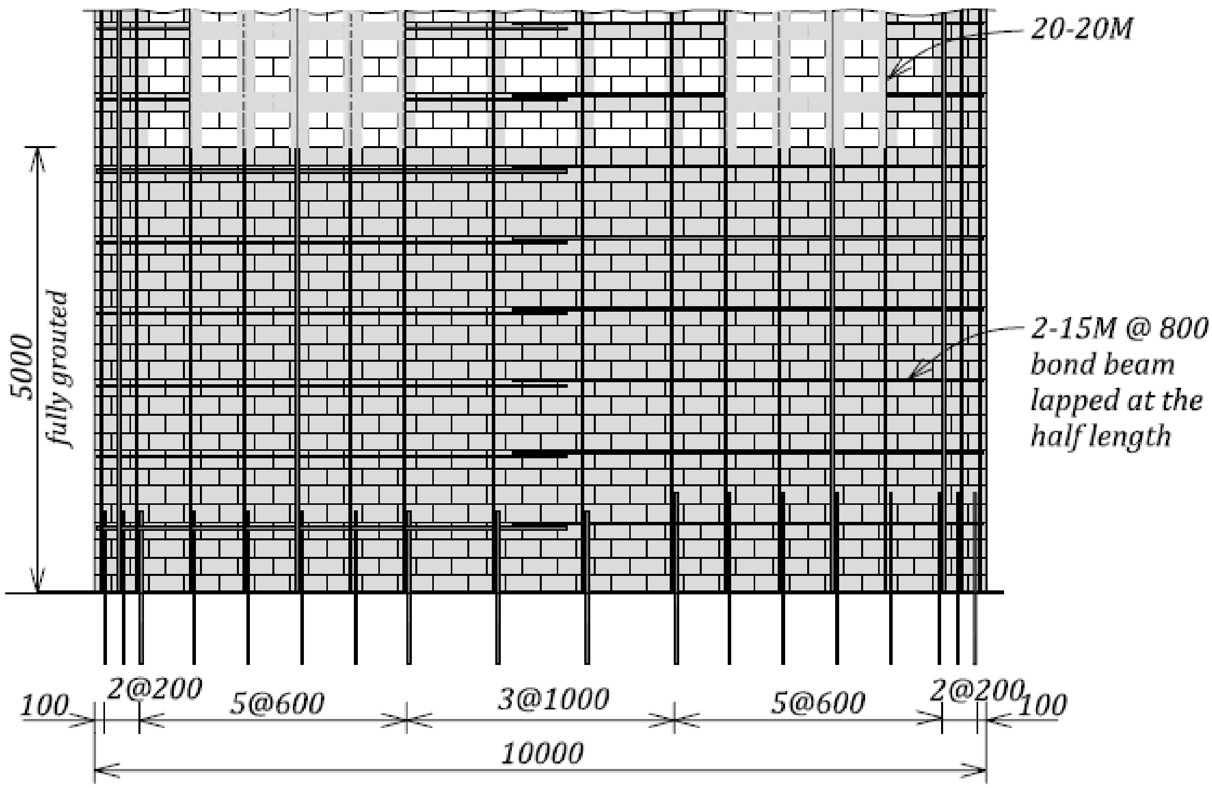
whichever is greater, from the end of the wall. In this case, the reinforcement should not be lapped within the distance c = 2400 mm from the end of the wall. The horizontal reinforcement can be lapped at the wall half-length.

3. Vertical reinforcement requirements (see Table 2-3).

There are no special detailing requirements for vertical reinforcement in limited ductility shear walls.

11. Design summary

Reinforcement arrangement for the wall under consideration is summarized on the figure below. Note that the shear wall of limited ductility must be solid grouted in plastic hinge region, but it may be partially grouted outside the plastic hinge region (this depends on the design forces).



12. Discussion

It is important to consider all possible behaviour modes and identify the one that governs in this design. The following three shear resistance values need to be considered:

a) $V_{nb} = 1718$ kN shear force corresponding to flexural failure

b) $V_r = 1769$ kN diagonal tension shear resistance

c) $V_r = 2196$ kN sliding shear resistance

Since the shear force corresponding to the flexural resistance is smallest of the three values, it can be concluded that the flexural failure mechanism is critical in this case, which is desirable for the seismic design.

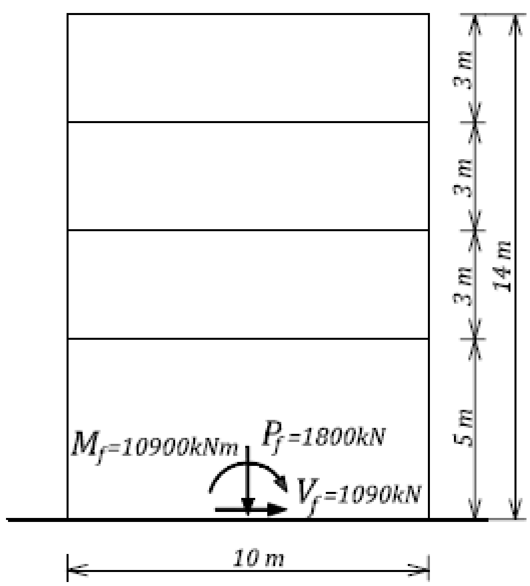
Had the design specified a shear wall of conventional construction, the same amount of vertical and horizontal reinforcement would have been required, but none of the special detailing discussed in step 10 would have been required. Also, the CSA S304.1 ductility check discussed in step 7 is not required for shear walls of conventional construction.

<u>EXAMPLE 5b</u>: Seismic design of a flexural shear wall of moderate ductility

Perform the seismic design of a shear wall X_1 located at the rear side of the building discussed in Example 2 and Example 5a. Try to use the same wall dimensions as in Example 5a, that is, 10 m length and 290 mm thickness.

The section at the base of the wall is subjected to the total dead load of 1800 kN (including the wall self-weight), the in-plane seismic shear force of 1090 kN, and the overturning moment of 10900 kNm. The design should meet the CSA S304.1 Cl.10.16.5 requirements for shear walls of moderate ductility.

Use hollow concrete blocks of 20 MPa unit strength and Type S mortar. Consider the wall as solid grouted. Grade 400 steel reinforcement (yield strength f_y = 400 MPa) is used for this design.



<u>SOLUTION:</u>

1. Material properties

Steel (both reinforcing bars and joint reinforcement):

ϕ_s = 0.85 f_y = 400 MPa

Masonry:

ϕ_m = 0.6

S304.1 Table 4, 20 MPa concrete blocks and Type S mortar:

f'_m = 10.0 MPa (assume solid grouted masonry)

2. Load analysis

The section at the base of the wall needs to be designed for the following load effects:

- P_f = 1800 kN axial load

- V_f = 1090 kN seismic shear force
- M_f = 10900 kNm overturning moment

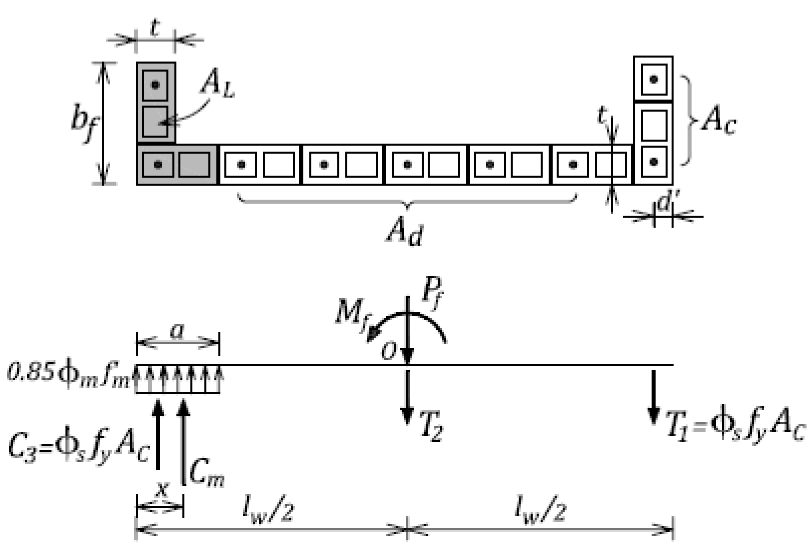
This is a moderate ductility shear wall, and NBCC 2005 Table 4.1.8.9 specifies the following R_d and R_o values (see Table 1-13):
R_d = 2.0 and R_o = 1.5
The seismic shear force of 1090 kN for a wall with moderate ductility ($R_d = 2.0$) was obtained by prorating the force of 1450 kN from Example 5a which corresponded to a shear wall with limited ductility ($R_d = 1.5$), as follows

$$V_f = 1450 * \frac{1.5}{2.0} \approx 1090 \text{ kN}$$

3. Height/thickness ratio check (Cl.10.16.5.2.2, see Section 2.5.4.4)
CSA S304.1-04 prescribes the following height-to-thickness (h/t) limit for the compression zone in <u>moderate</u> ductility shear walls:
$h/(t+10) < 14$
For this example,
h = 5000 mm (the largest unsupported wall height)
So,
$t \geq h/14 - 10 = 347$ mm
This exceeds the maximum possible wall thickness of 290 mm, which was used in Example 5a.

Commentary to Section 2.5.4.4 contains the following two alternative approaches for verifying the out-of-plane stability of ductile masonry shear walls:

1) Provide flanges at the wall ends and prove that the out-of-plane stability of the compression zone is satisfactory.
Try the effective flange width
$b_f = 690$ mm
The wall section and the internal force distribution is shown on the figure.



This procedure assumes that the concentrated reinforcement (area A_c) is provided at the wall ends, while the remaining reinforcement (area A_d) is distributed over the wall length. After a few trial estimates, the total area of vertical reinforcement A_{vt} was determined as follows

A_{vt} = 2800 mm^2

Concentrated reinforcement area (2-15M bars at each wall end):

A_c = 400 mm^2

Distributed reinforcement area:

A_d =2800-2*400=2000 mm^2

Distance from the wall end to the centroid of concentrated reinforcement A_c:

$d' = 145$ mm

- *Check the buckling resistance of the compression zone.*

The area of the compression zone A_L:

$$A_L = \frac{P_f + \phi_s f_y A_d}{0.85\phi_m f'_m} = \frac{1800*10^3 + 0.85*400*2000}{0.85*0.6*10.0} = 4.86*10^5$$

The depth of the compression zone a:

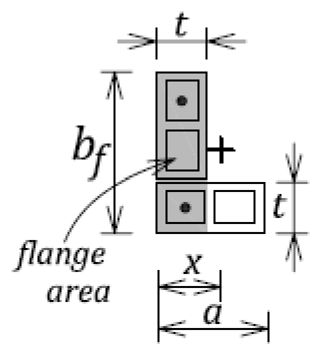
$$a = \frac{A_L - b_f * t + t^2}{t} = \frac{4.86*10^5 - (690*290) + 290^2}{290} = 1276 \text{ mm}$$

The neutral axis depth:

$$c = \frac{a}{0.8} = 1595 \text{ mm}$$

The centroid of the masonry compression zone:

$$x = \frac{t*\left(a^2/2\right) + (b_f - t)\left(t^2/2\right)}{A_L} = 520 \text{ mm}$$



In this case, the compression zone is L-shaped, however only the flange area will be considered for the buckling resistance check (see the shaded area shown on the figure). This is a conservative approximation and it is considered to be appropriate for this purpose, since the gross moment of inertia is used.

Gross moment of inertia for the flange only:

$$I_{xg} = \frac{t*b_f^{\ 3}}{12} = \frac{290*690^3}{12} = 7.94*10^9 \text{ mm}^4$$

The buckling strength for the compression zone will be determined according to S304.1 Cl. 10.7.4.3, as follows:

$$P_{cr} = \frac{\pi^2 \phi_{er} E_m I}{(1+0.5\beta_d)(kH)^2} = 19983kN$$

where

$\phi_{er} = 0.75$

$k = 1.0$ pin-pin support conditions

$\beta_d = 0$ assume 100% seismic live load

$H = 5000$ mm unsupported wall height

$E_m = 850 f'_m = 8500$ MPa modulus of elasticity for masonry

- *Find the resultant compression force (including the concrete and steel component).*

$P_{fb} = C_m + \phi_s f_y A_c = 2480*10^3 + 0.85*400*400 = 2620$ kN

where

$C_m = (0.85\phi_m f'_m)A_L = (0.85*0.6*10.0)(4.86*10^5) = 2480$ kN

- *Confirm that the out-of-plane buckling resistance is adequate.*

Since

$P_{fb} = 2620$ kN < $P_{cr} = 19983$ kN

it can be concluded that the out-of-plane buckling resistance is adequate. The flanged section can be used for this design.

2) Prove that the compression zone of the wall is small and the adjacent vertical strips are able to stabilize it.

For flanged wall sections, the neutral axis depth needs to meet one of the following requirement (see Figure 2-28c):

$c^* \le 6t = 6*290 = 1740$ mm

Note that $6t$ denotes the distance from the inside of a wall flange to the point of zero strain. So, the total neutral axis depth (distance from the extreme compression fibre to the point of zero strain) is equal to

$c = c^* + t = 1740 + 290 = 2030$ mm

The neutral axis depth was determined above, as follows

$c = 1595$ mm < 2030 mm

Based on these two checks, the out-of-plane wall stability should not be a concern when a flanged section is used for the design, and so the CSA S304.1 height-to-thickness restrictions for moderate ductility shear walls will be relaxed.

4. Design the flanged section for the combined axial load and flexure – consider distributed and concentrated wall reinforcement (see Section C.1.1.1).

The key design parameters for this calculation were determined in step 3 above. The factored moment resistance M_r will be determined by summing up the moments around the centroid of the wall section as follows

$$M_r = C_m(l_w/2 - x) + 2(\phi_s f_y A_c)(l_w/2 - d') = 2480*10^3*(10000/2 - 520) + 2*(0.85*400*400)*(10000/2 - 145)$$

$M_r = 12431$ kNm > $M_f = 10900$ kNm   OK

5. Perform the CSA S304.1 ductility check (see Section 2.5.4.3).

To satisfy the CSA S304.1 ductility requirements for <u>moderate ductility</u> shear walls (Cl.10.16.5.2.3), neutral axis depth ratio (c/l_w) should be less than the following limit:

$c/l_w < 0.2$ when $h_w/l_w < 4$

In this case,

$h_w/l_w = 14000/10000 = 1.4 < 4$ and $c = 1595mm$

thus

$c/l_w = 1595/10000 = 0.16 < 0.2$

Therefore, the CSA S304.1 ductility requirement is satisfied.

6. The diagonal tension shear resistance (see Section 2.3.2 and CSA S304.1 Cl.10.10.1)
Masonry shear resistance (V_m):

$b_w = 290$ mm overall wall thickness

$d_v \approx 0.8 l_w = 8000$ mm effective wall depth

$\gamma_g = 1.0$ solid grouted wall

$P_d = 0.9 P_f$ = 1620 kN

$$v_m = 0.16(2 - \frac{M_f}{V_f d_v})\sqrt{f'_m} = 0.51 \text{ MPa}$$

$$\frac{M_f}{V_f d_v} = \frac{10900}{1090 * 8.0} = 1.25 > 1.0$$

$$V_m = \phi_m (v_m b_w d_v + 0.25 P_d)\gamma_g = 0.6(0.51*290*8000+0.25*1620*10^3)*1.0 = 953 \text{ kN}$$

To find the steel shear resistance V_s, assume 2-15M bond beam reinforcing bars at 600 mm spacing (this should make some allowance in the shear strength to satisfy capacity design), thus

$A_v = 400$ mm^2

$s = 600$ mm

$$V_s = 0.6\phi_s A_v f_y \frac{d_v}{s} = 0.6 * 0.85 * \frac{400}{1000} * 400 * \frac{8000}{600} = 1088 \text{ kN}$$

According to Cl.10.16.5.3.1, there is a 50% reduction in the masonry shear resistance contribution for moderate ductility shear walls, and so

$V_r = 0.5 V_m + V_s = 0.5 * 953 + 1088 = 1565$ kN $> V_f = 1090$ kN OK

Maximum shear allowed on the section is (S304.1 Cl.10.10.1.1)

$\max V_r = 0.4 \phi_m \sqrt{f'_m} b_w d_v \gamma_g = 1760$ kN $> V_r$ OK

7. Capacity design check (see Section 2.5.2)
At this point, both the moment resistance M_r and the diagonal shear resistance V_r for the wall section have been determined. S304.1 Cl.10.16.3.3 requires that ductile reinforced masonry shear walls be designed so that flexural failure takes place before shear failure has been initiated, that is, to follow the capacity design approach (see Section 2.5.2 for more details).

In this case, the factored moment resistance is equal to

$M_r = 12431$ kNm

The nominal moment resistance can be estimated as follows

$$M_n = \frac{M_r}{\phi_s} = \frac{12431}{0.85} = 14625 \text{ kNm}$$

Shear force acts at the effective height h_e, that is, distance from the base of the wall to the resultant of all seismic forces acting at floor levels. h_e can be determined as follows

$$h_e = \frac{M_f}{V_f} = 10.0 \text{ m}$$

The shear force V_{nb} that would cause the overturning moment equal to M_n is as follows

$$V_{nb} = \frac{M_n}{h_e} = \frac{14625}{10.0} = \text{ 1528 kN } < V_r = 1463 \text{ kN OK}$$

8. Shear resistance at the web-to-flange interface (Cl.7.11.4, see Section C.2).

The factored shear stress at the web-to-flange interface is equal to the larger of horizontal and vertical shear stress, as shown below.

Horizontal shear can be determined as follows:

$$v_f = \frac{V_f}{t_e l_w} = \frac{1090*10^3}{290*10000} = 0.38 \text{ MPa}$$

where t_e = 290 mm (effective wall thickness)

Vertical shear over the entire wall height (caused by the resultant compression force P_{fb} calculated in Step 3):

$$v_f = \frac{P_{fb}}{b_w * h_w} = \frac{2620*10^3}{290*14000} = 0.64 \text{ MPa} \qquad \text{governs}$$

Masonry diagonal tension shear strength:

$v_m = 0.51$ MPa (see step 6)

Since

$v_f = 0.64$ MPa > $\phi_m v_m = 0.31$ MPa

it is required to provide shear reinforcement at the web-to-flange interface. Since the horizontal reinforcement consists of 2-15M bars @ 600 mm spacing (bond beam reinforcement); both bars can be extended into the flange (90° hook), and so

$$v_s = \frac{\phi_s A_s f_y}{s \cdot t_e} = \frac{0.85*2*200*400}{600*290} = 0.78 \text{ MPa}$$

The total shear resistance

$v_r = \phi_m v_m + v_s = 0.31 + 0.78 = 1.09$ MPa > $v_f = 0.64$ MPa OK

9. Sliding shear resistance (see Section 2.5.4.6)

The factored in-plane sliding shear resistance V_r is determined as follows:

μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

A_s = 2800 mm^2 total area of vertical wall reinforcement

For moderate ductility shear walls, only the vertical reinforcement in the tension zone should be accounted for in the T_y calculations (Cl.10.16.5.3.2), and so (see Figure 2-17b)

$$T_y = \phi_s A_s f_y \left(\frac{l_w - c}{l_w} \right) = 0.85*2800*400*\left(\frac{10000-1595}{10000} \right) = 800 \text{ kN}$$

P_d = 1620 kN

$P_2 = P_d + T_y$ = 1620+800 = 2420 kN

$V_r = \phi_m \mu P_2$ = 0.6*1.0*2420 = 1452 kN

$V_r = 1452$ kN > $V_f = 1090$ kN OK

10. CSA S304.1 seismic detailing requirements for moderate ductility walls – plastic hinge region

According to Cl.10.16.5.2.1, the required height of the plastic hinge region for moderate ductility shear walls must be greater than (see Table 2-4)

$l_p = l_w = 10.0$ m

or

$l_p = h_w / 6 = 14.0 / 6 = 2.3$ m

(note that h_w denotes the total wall height)

So, $l_p = 10.0$ m governs

Reinforcement detailing requirements for the plastic hinge region of limited ductility shear walls are as follows:

1. ***The wall in the plastic hinge region must be solid grouted (Cl.10.16.4.1.3, see Table 2-4).***
2. ***Horizontal reinforcement requirements (see Figure 2-31)***

   a) Reinforcement spacing should not exceed the following limits (Cl.10.16.4.3.3), see Table 2-2:

   $s \le 1200$ mm or

   $s \le l_w/2 = 10000/2 = 5000$ m

   Since the lesser value governs, the maximum permitted spacing is

   $s \le 1200$ mm

   According to the design (see step 7), the horizontal reinforcement spacing is 600 mm, hence OK.

   b) Detailing requirements (see Table 2-3)

   Horizontal reinforcement shall not be lapped within (Cl.10.16.4.3.3)

   600 mm or

   c = 1595 mm (the neutral axis depth)

   whichever is greater, from the end of the wall. In this case, the reinforcement should not be lapped within the distance c = 1595 mm from the end of the wall. The horizontal reinforcement can be lapped at the wall half-length.

   Horizontal reinforcement shall be (Cl.10.16.5.4.2):

   i) provided by reinforcing bars only (no joint reinforcement!);

   ii) continuous over the length of the wall (can be lapped in the centre), and

   iii) have 180° hooks around the vertical reinforcing bars at the ends of the wall.

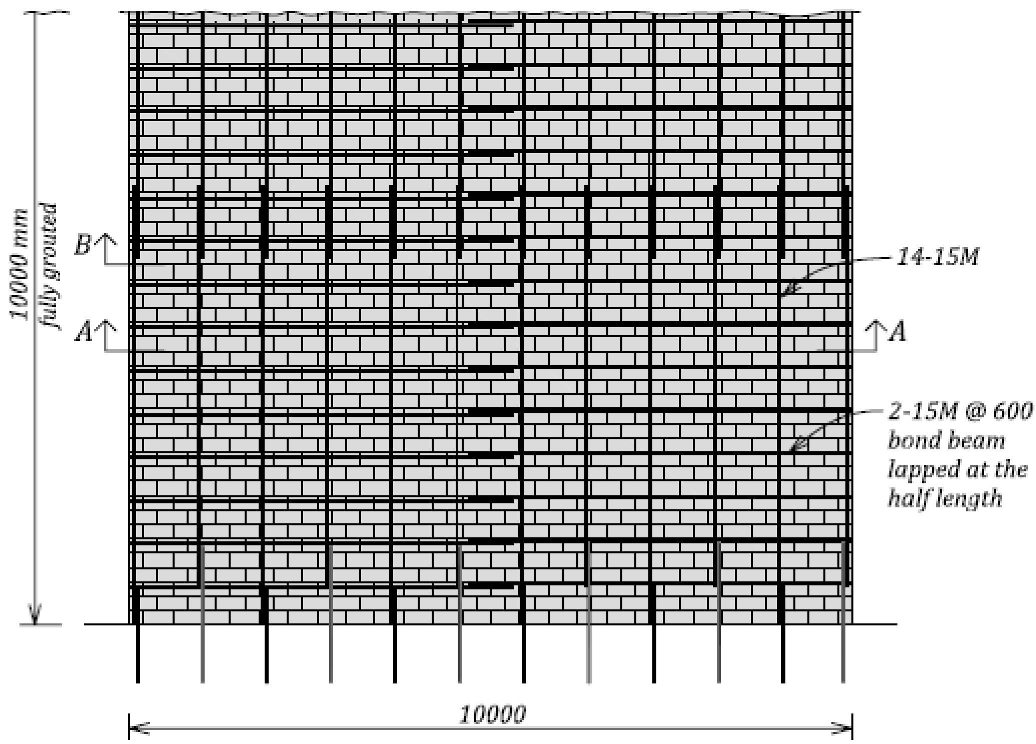
   All these requirements will be complied with, as shown on the design summary drawing.

3. Vertical reinforcement requirements (Cl.10.16.5.4.1)

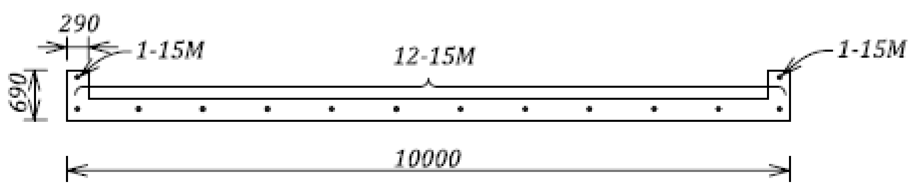
At any section within the plastic hinge region, no more than half of the area of vertical reinforcement may be lapped (see Table 2-3 and Figure 2-31).

11. Design summary

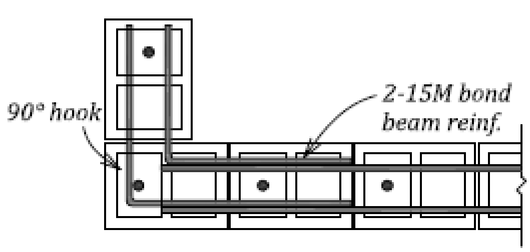
Reinforcement arrangement for the wall under consideration is summarized on the next page. Note that the shear wall of limited ductility must be solid grouted in plastic hinge region, but it may be partially grouted outside the plastic hinge region (this depends on the design forces).



Elevation



Section A-A



Section B-B

12. Discussion

It is important to consider all possible behaviour modes and identify the one that governs in this design. The following three shear resistance values need to be considered:

a) $V_{nb} = 1463$ kN shear force corresponding to flexural failure

b) $V_r = 1565$ kN diagonal tension shear resistance

c) $V_r = 1452$ kN sliding shear resistance

Since the sliding shear resistance value is smallest, it can be concluded that the sliding shear mechanism is critical in this case. Sliding shear resistance can be increased by roughening the wall-to-foundation interface (in which case the frictional coefficient can be increased to μ = 1.4) or by providing shear keys. Alternatively, additional dowels could be provided at the base of the wall, however this would result in an increase in the moment resistance. The designer would need to ensure that the capacity design criterion discussed in step 7 is satisfied.

At this point, it is of interest to compare the designs for limited ductility shear wall (Example 5a) and the moderate ductility shear wall (Example 5b). The walls are very similar, and have the same height, length, and thickness. The wall from this example has a flanged section, while the wall from Example 5a has a rectangular section. The walls are subjected to the same seismic hazard, but differ depending on the R_d and R_o values required for the respective designs. The comparison of the two designs is presented in the table below.

Table 1. A Comparison of the Limited Ductility and Moderate Ductility Shear Wall Designs

	Limited ductility shear wall (Example 5a)	**Moderate ductility shear wall (Example 5b)**
Actual height/thickness ratio for the plastic hinge zone (CSA S304.1 limit)	17.2 (18)	17.2 (14) – flanges used to stabilize compression zones at wall ends
Vertical reinforcement	6000 mm^2 (20-20M)	2800 mm^2 (14-15M)
Horizontal reinforcement	2-15M@800 mm bond beam reinforcement	2-15M@600 mm bond beam reinforcement
Plastic hinge length l_p	5 m	10 m
Horizontal reinforcement detailing (Table 2-3)	Minimal requirements	More extensive detailing requirements (see step 10)
Vertical reinforcement detailing (Table 2-3)	No special requirements	Lapping requirement (see step 10)

The key differences in these designs can be summarized as follows:

1. Moderate ductility shear wall requires less vertical reinforcement (by approximately 50%) since the dead load provides proportionally more of the moment resistance.

2. Moderate ductility shear wall requires more horizontal reinforcement (by approximately 30%) as only 50% of the masonry shear strength can be utilized.

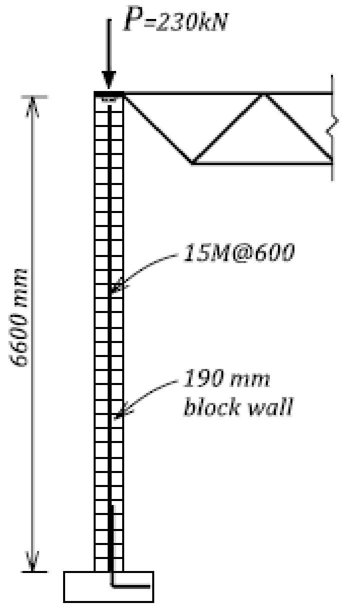
3. Moderate ductility shear wall requires a substantially larger plastic hinge zone (10 m high) as compared to the limited ductility shear wall (5 m high) – the wall needs to be solid grouted and special detailing requirements apply in this zone, although the limited ductility shear wall may also need to be solid grouted for a height greater than 5 m.

4. Moderate ductility shear wall has more extensive horizontal and vertical reinforcement detailing requirements.

5. Moderate ductility shear wall requires flanges in order to satisfy the CSA S304.1 height/thickness requirements.

EXAMPLE 6 a: Design of a loadbearing wall for out-of-plane seismic effects

Verify the out-of-plane seismic resistance of the loadbearing block wall designed for in-plane loads in Example 4b, according to NBCC 2005 and CSA S304.1 requirements. The wall is a part of a single-storey warehouse building located in Ottawa, ON, with soil corresponding to Site Class C. The wall is 8 m long and 6.6 m high, and is subjected to a total dead load of 230 kN (including its self-weight). The wall is constructed with 200 mm hollow concrete blocks of 15 MPa unit strength, Type S mortar, and solid grouting. The wall is reinforced with 15M Grade 400 vertical rebars at 600 mm on centre spacing. The slenderness effects outlined in CSA S304.1 will not be considered in this design.



SOLUTION:

1. Material properties

Steel (both reinforcing bars and joint reinforcement):

ϕ_s = 0.85 f_y = 400 MPa

Masonry:

ϕ_m = 0.6

S304.1 Table 4, 15 MPa concrete blocks and Type S mortar:

f'_m = 7.5 MPa (assume solid grouted masonry)

2. Determine the out-of-plane seismic load according to NBCC 2005 (see Section 2.6.5.3).

This design requires the calculation of seismic load V_p for parts of buildings and nonstructural components according to NBCC 2005 Cl.4.1.8.17. First, seismic design parameters need to be determined as follows:

- Location: Ottawa, ON

  $S_a(0.2)$ = 0.66 (NBCC 2005 Appendix C, page C-22)

- Foundation factors

  F_a = 1.0 for $S_a(0.2)$ =0.66 and Site Class C (from Table 1-10 or NBCC 2005 Table 4.1.8.4.B)
- I_E = 1.0 normal importance building

Find S_p (horizontal force factor for part or portion of a building and its anchorage per NBCC 2005, Table 4.1.8.17, Case 1)

$C_p = 1.0 \quad A_r = 1.0 \quad R_p = 2.5 \quad A_x = 3.0$ ($h_x = h_n$ top floor)

$S_p = C_p A_r A_x / R_p = 1.0 \cdot 1.0 \cdot 3.0 / 2.5 = 1.2$

$0.7 < S_p < 4.0$ O.K.

- W_p = 4.0 kN/m^2 unit weight of the 190 mm block wall (solid grouted)

Seismic load V_p can be calculated as follows:

$V_p = 0.3 F_a S_a(0.2) I_E S_p W_p$ =0.3*1.0*0.66*1.0*1.2*(4.0 kN/m^2) = 0.95 kN/m^2 ≈ 1.0 kN/m^2

3. Determine the effective compression zone width (b) for the out-of-plane design (see Section 2.4.2).

According to S304.1 Cl.10.6.1, the effective compression zone width (b) should be taken as the lesser of the following two values (see Figure 2-19):

$b = s = 600$ mm spacing of vertical reinforcement

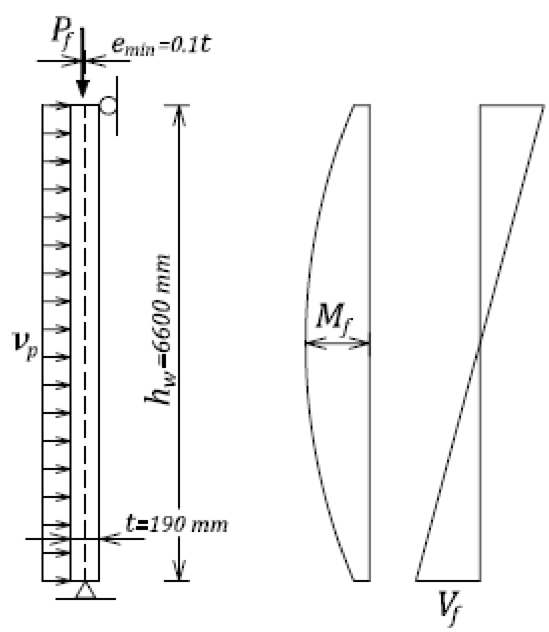
or

$b = 4t = 4*190 = 760$ mm

All design calculations in this example will be performed considering a vertical wall strip of width $b = 600$ mm.

4. Find the design shear force and the bending moment.

The wall will be modeled as a simple beam with pin supports at the base and top. The loads on the wall consist of axial load due to roof load and wall self-weight, plus the seismic out-of-plane load. The roof load and wall self-weight create moments due to minimum axial load eccentricity.



- Axial load per wall width equal to $b = 600$ mm:

$$P_f = \frac{P}{l_w} * b = \frac{230kN}{8m} * 0.6 = 17.25 \approx 17.0 \text{ kN}$$

- Minimum eccentricity (S304.1 Cl.10.7.2)

$e_{\min} = 0.1t =$ 0.019 m

- Out-of-plane seismic load per wall width equal to $b = 600$ mm:

$v_p = 1.0 * 0.6 = 0.6$ kN/m

- Design bending moment (at the midheight):

$$M_f = p * e_{\min} + \frac{v_p * h_w^2}{8} = 17 * 0.019 + \frac{0.6 * 6.6^2}{8}$$

= 3.59 ≈ 3.6 kNm

5. Check whether the wall resistance for the combined effect of axial load and bending is adequate (see Section C.1.2).
This can be verified from a P-M interaction diagram which can be developed using the EXCEL© software (or commercially available masonry design software). Relevant tables used to develop the diagram are presented below, while the detailed theoretical background is outlined in Section C.1.2. Note that the design width is equal to $b = 600mm$.

Table 1. Design Parameters

Design parameter	Unit	Symbol	Value
Wall thickness	mm	t	190
Design width	mm	b	600
Masonry maximum strain		EPSm	0.003
Masonry strength	MPa	f'm	7.5
Steel yield strength	MPa	fy	400
Steel modulus of elasticity	MPa	Es	200000
Effective depth	mm	d	95
(c/d)balanced			0.6
Reinforcement area	mm^2/b	As	200
Material resistance-masonry		Fim	0.6
Material resistance-steel		Fis	0.85
X- factor		X	1
BETA1		BETA1	0.8
Effective area	mm^2	Ae	114000

In this case, the reinforcement is placed at the centre of the wall and so

$$d = \frac{t}{2} = \frac{190}{2} = 95 \text{ mm}$$

The neutral axis depth corresponding to a balanced condition (onset of yielding in the steel and maximum compressive strain in masonry) can be determined from the following proportion

$$\frac{c_b}{d - c_b} = \frac{\varepsilon_m}{\varepsilon_y}$$

For $\varepsilon_m = 0.003$ and $\varepsilon_y = 0.002$ it follows that

$$c_b = 0.6d$$

The area of vertical reinforcement per width $b = 600$ mm can be determined as follows:

$$A_s = \frac{A_b}{s} * b = \frac{200}{600} * 600 = 200 \text{ mm}^2 \quad \text{(15M@ 600 mm reinforcement)}$$

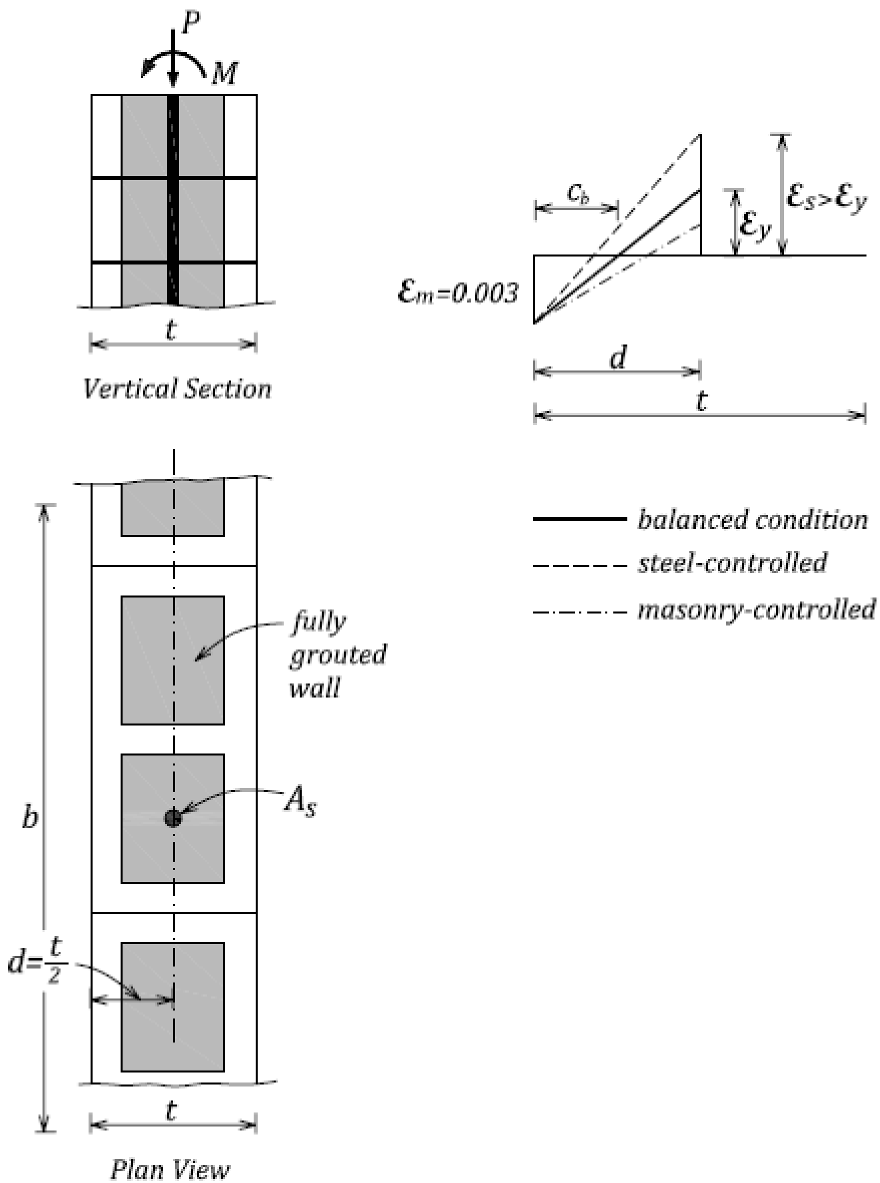
To determine whether the wall can carry the combined effect of axial load and bending moment, it is useful to construct an axial load-moment interaction diagram (also known as P-M interaction diagram). The P-M interaction diagram for this example was developed using Microsoft

EXCEL® spreadsheet, but other methods or computer programs are also available. The results of the calculations are presented in Table 2.

Table 2. P-M Interaction Diagram Values

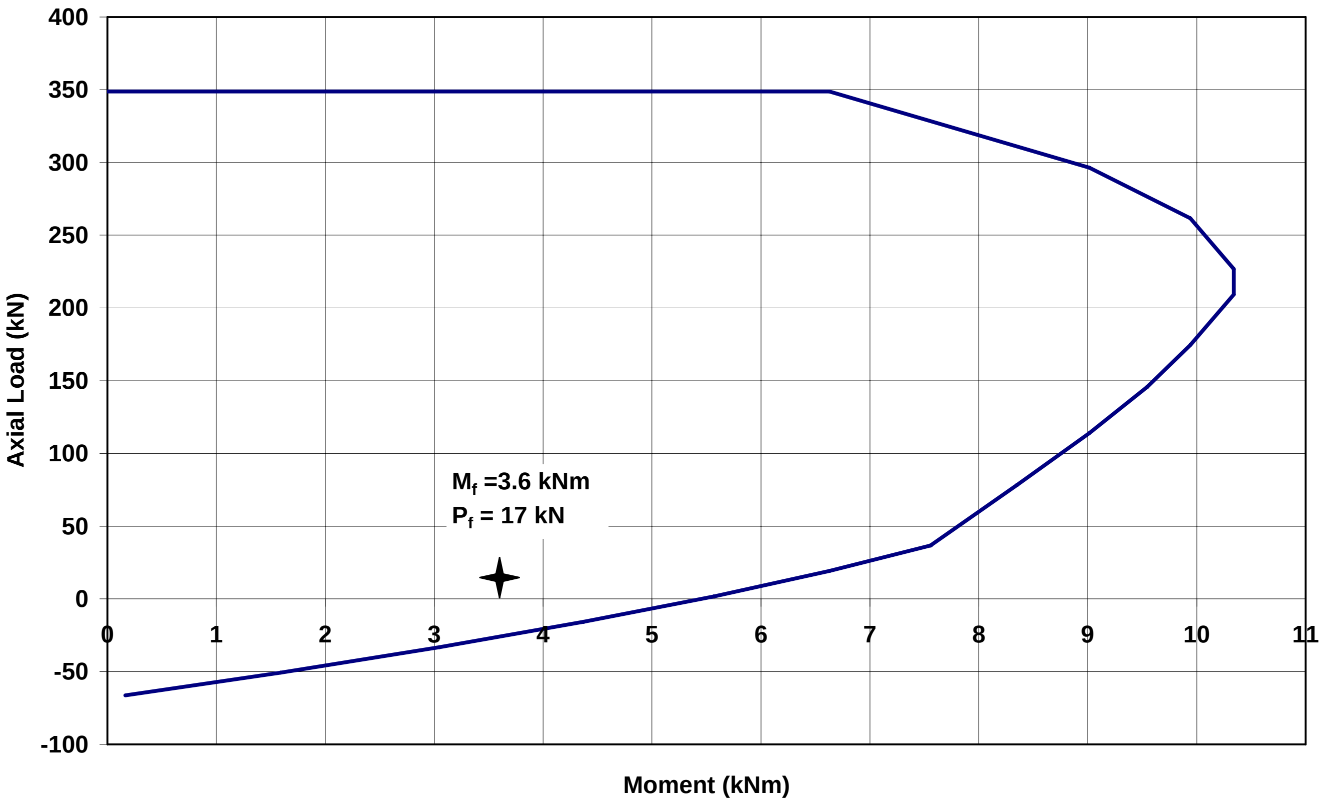
		c/d	c	C_m	EPSs	T_r	M_r	P_r
			mm	N		N	kNm	kN
Points controlled by steel $c<c_b$		0.01	0.95	1744.2	0.02	68000	0.16504	-66.256
		0.1	9.5	17442	0.02	68000	1.59071	-50.558
		0.2	19	34884	0.02	68000	3.04886	-33.116
		0.3	28.5	52326	0.02	68000	4.37445	-15.674
		0.4	38	69768	0.02	68000	5.56749	1.768
		0.5	47.5	87210	0.02	68000	6.62796	19.21
		0.6	57	104652	0.02	68000	7.55587	36.652
Points controlled by masonry $c>c_b$		0.6	57	104652	0.002	68000	7.55587	36.652
		0.7	66.5	122094	0.00129	43714.3	8.35123	78.3797
		0.8	76	139536	0.00075	25500	9.01403	114.036
		0.9	85.5	156978	0.00033	11333.3	9.54426	145.645
Full section under compression		1	95	174420	0	0	9.94194	174.42
		1.2	114	209304	-0.0005	-17000	10.3396	209.304
		1.3	123.5	226746	-0.0007	-23538	10.3396	226.746
		1.5	142.5	261630	-0.001	-34000	9.94194	261.63
		1.7	161.5	296514	-0.0012	-42000	9.01403	296.514
		2	190	348840	-0.0015	-51000	6.62796	348.84
Pure compression							0	348.84

The three basic cases considered in the development of the interaction diagram (steel-controlled behaviour, masonry-controlled behaviour, and the balanced condition) are illustrated on the figure below. For more detailed explanation related to the development of P-M interaction diagrams refer to Section C.1.2.



The P-M interaction diagram showing the point of interest ($M_f = 3.6$ kNm and $P_f = 17$ kN) is shown below. It is obvious that the wall resistance to combined effects of axial load and out-of-plane bending is adequate for the given design loads and the reinforcement determined in Example 4b.

Wall P-M Interaction Diagram



6. Check whether the out-of-plane shear resistance of the wall is adequate (Cl.10.10.2, see Section 2.4.2).

Design shear force at the support per wall width $b = 600$ mm:

$$V_f = \frac{v_p * h_w}{2} = \frac{0.6 * 6.6}{2} \approx 2.0 \text{ kN}$$

According to S304.1 Cl.10.10.2, the factored out-of-plane shear resistance (V_r) shall be taken as follows

$$V_r = \phi_m (v_m \cdot b \cdot d + 0.25 P_d)$$

where

$v_m = 0.16\sqrt{f'_m}$ = 0.44 MPa (f'_m = 7.5 MPa for solid grouted 15 MPa block)

$d = 95$ mm effective depth (to the block mid-depth)

$b = 600$ mm effective compression zone width

The axial load P_d can be determined as

$$P_d = 0.9 P_f = 0.9 * 17.25 = 15.5 \text{ kN}$$

(note that the load has been prorated in proportion to the effective compression zone width b).

So,

$$V_r = 0.6 * (0.44 * 600 * 95 + 0.25 * 15500) = 17.4 \text{ kN}$$

Since

$V_f = 2.0$ kN < $V_r = 17.4$ kN OK

Maximum shear allowed on the section is

$\max V_r = 0.4\phi_m \sqrt{f'_m}(b*d) = 0.4*0.6*\sqrt{7.5}*(600*95) = 37.5$ kN OK

7. Check the sliding shear resistance (see Section 2.4.3).

The factored out-of-plane sliding shear resistance V_r is determined according to S304.1 Cl.10.10.4.2, as follows:

μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

A_s = 200 mm^2 area of vertical reinforcement per wall width $b = 600$ mm

$T_y = \phi_s A_s f_y$ = 0.85*200*400 = 68 kN

$P_d = 0.9P_f = 15.5$ kN

$P_2 = P_d + T_y$ = 15.5+68 = 83.5 kN

$V_r = \phi_m \mu P_2$ = 0.6*1.0*83.5= 50.0 kN

$V_r = 50.0$ kN > $V_f = 2.0$ kN OK

Note that the sliding shear resistance does not govern in this case, however this mechanism often governs the in-plane shear resistance.

8. Conclusion

It can be concluded that the out-of-plane seismic resistance for this wall is satisfactory. This wall seems to be overdesigned for the out-of-plane resistance because the in-plane seismic design governs (this is a common scenario in design practice).

EXAMPLE 6 b: Design of a nonloadbearing wall for out-of-plane seismic effects

Consider the same masonry wall discussed in Example 6a, but in this example treat is as a nonloadbearing wall. The wall is 8 m long and 6.6 m high and is constructed using 200 mm hollow concrete blocks of 15 MPa unit strength and Type S mortar. Verify the out-of-plane seismic resistance of the wall according to NBCC 2005 and CSA S304.1 seismic requirements.

Consider the following two cases:
a) unreinforced wall, and
b) reinforced partially grouted wall (use Grade 400 steel reinforcement for this design).

Use the seismic load determined in Example 6a, that is, $v_p = 1.0$ kN/m^2.

SOLUTION:

Material properties
Steel (both reinforcing bars and joint reinforcement):
ϕ_s = 0.85 f_y = 400 MPa
Masonry:
ϕ_m = 0.6
Compresion resistance (S304.1 Table 4, 15 MPa concrete blocks and Type S mortar):
f'_m = 9.8 MPa (ungrouted, or partially grouted ignoring grout area)
Tension resistance normal to bed joint (S304.1 Table 5):
f_t = 0.4 MPa (ungrouted)

Find the design shear force and the bending moment.
The wall will be modeled as a simple beam with pin supports at the base and the top. The wall height is $h_w = 6.6$ m. A unit wall strip (width $b = 1000$ mm) will be considered for this design.

The forces on the wall consist of the axial load due to the wall self-weight and the bending moment due to seismic out-of-plane load (NBCC 2005 load combination 1xD+1xE).

- Factored axial load per width b of 1.0 m:

wall weight w = 2.46 kN/m^2 (ungrouted 190 mm block wall)

$$P_f = w * \frac{h_w}{2} * b = (2.46) * \frac{6.6}{2} * 1.0 = 8.1 \text{ kN/m}$$

- Out-of-plane seismic load per width b of 1.0 m:

$v_p = 1.0$ kN/m

- Factored bending moment (at the midheight):

$$M_f = \frac{v_p * h_w^2}{8} = \frac{1.0 * 6.6^2}{8} \approx 5.5 \text{ kNm/m}$$

- Factored shear force (at the support):

$$V_f = \frac{v_p * h_w}{2} = \frac{1.0 * 6.6}{2} \approx 3.3 \text{ kN/m}$$

a) Unreinforced wall

Check whether the wall resistance to the combined effect of axial load and bending is adequate (see Section 2.6.1.3).

Find the load eccentricity:

$$e = \frac{M_f}{P_f} = \frac{5.5kNm}{8.1kN} = 0.68m = 680mm$$

According to S304.1 Cl.7.2.1, an unreinforced masonry wall is to be designed as uncracked if $e > 0.33t$

where t denotes the wall thickness ($t = 190mm$)

$0.33t = 0.33*190 = 63mm$

In this case,

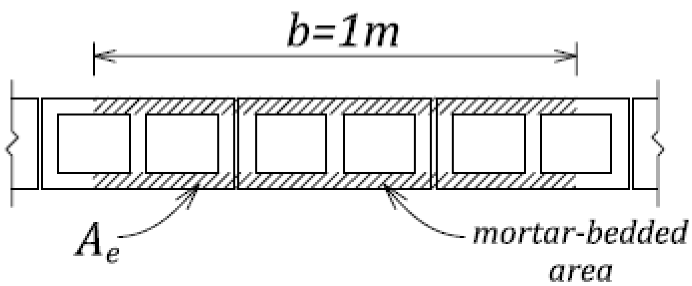
$e = 680mm > 0.33t = 63mm$

so the wall will be designed as uncracked (i.e. the maximum tensile stress is less than the allowable value) according to S304.1 Cl.7.2.2. The design procedure is explained in Section 2.6.1.3.

First, we need to determine properties for the effective wall section for a width $b = 1000$ mm. For a hollow 190 mm wall, the values obtained from Table D-1 are as follows:

$A_e = 75.4*10^3$ mm^2/m effective cross-sectional area

$S_e = 4.66*10^6$ mm^3/m section modulus of effective cross-sectional area



The maximum compression stress at the wall face can be calculated as follows:

$$\max\ f_c = \frac{P_f}{A_e} + \frac{M_f}{S_e} = \frac{8.1*10^3}{75.4*10^3} + \frac{5.5*10^6}{4.66*10^6} = 0.107 + 1.18 = 1.29MPa$$

The allowable value is equal to

$\phi_m f'_m = 0.6*9.8 = 5.9MPa$

Since

$\max\ f_c = 1.29MPa < 5.9MPa$

it follows that the maximum compression stress is less than the allowable value.

Find the maximum tensile stress as follows:

$$\max\ f_t = \frac{P_f}{A_e} - \frac{M_f}{S_e} = \frac{8.1*10^3}{75.4*10^3} - \frac{5.5*10^6}{4.66*10^6} = 0.107 - 1.18 = -1.07MPa$$

The allowable value is equal to

$-\phi_m f_t = -0.6*0.4 = -0.24MPa$

Since

$\max f_t = -1.07MPa < -0.24MPa$

it follows that the maximum tensile stress exceeds the allowable value, which is not acceptable.

In this design, the tensile stress criterion is not going to be satisfied even if the wall thickness is increased to 290 mm. Therefore, a reinforced masonry wall is required in this case. Also, reinforcement in this wall is mandatory since the wall is to be constructed at Ottawa, ON, where the seismic hazard index $I_E F_a S_a(0.2)$=1.0*1.0*0.66=0.66 > 0.35. Therefore, the design will proceed considering a reinforced nonloadbearing wall.

b) Reinforced wall

1. Find the minimum seismic reinforcement for nonloadbearing walls (see Section 2.6.4).

According to S304.1 Cl.10.15.2.4, if $0.35 \le I_E F_a S_a(0.2) \le 0.75$ nonloadbearing walls shall be reinforced in one or more directions with reinforcing steel having a minimum total area of

$A_{stotal} = 0.0005A_g$

The reinforcement may be placed in one direction, provided that it is located to reinforce the wall adequately against lateral loads and spans between lateral supports.

$A_{stotal} = 0.0005A_g$ = 0.0005*(190*10^3 mm^2) = 95 mm^2/m

where

A_g =(1000mm)*(190mm)=190*10^3 mm^2 gross cross-sectional area per metre of wall length

Let us choose 15M vertical reinforcement (area 200 mm^2) at 1200 mm spacing which is the maximum spacing allowed (1200 mm).

The area of reinforcement per metre of wall length is

$A_s = 200*\frac{1000}{1200} = 167$ mm^2/m > 95 mm^2/m OK

2. Determine the effective compression zone width (b) for the out-of-plane design (see Section 2.4.2).

The wall resistance will be determined considering a strip equal to the bar spacing s=1200 mm, as follows:

$P_f = 8.1*\frac{1.2}{1.0} = 9.7$ kN

$M_f = 5.5*\frac{1.2}{1.0} = 6.6$ kNm

$V_f = 3.3*\frac{1.2}{1.0} = 4.0$ kN

3. Check whether the wall resistance to the combined effect of axial load and bending is adequate (see Section C.1.2).

Since this is a partially grouted wall, its flexural resistance will be determined using a T-section model.

According to S304.1 Cl.10.6.1, the effective compression zone width (b) should be taken as the lesser of the following two values (see Figure 2-19):

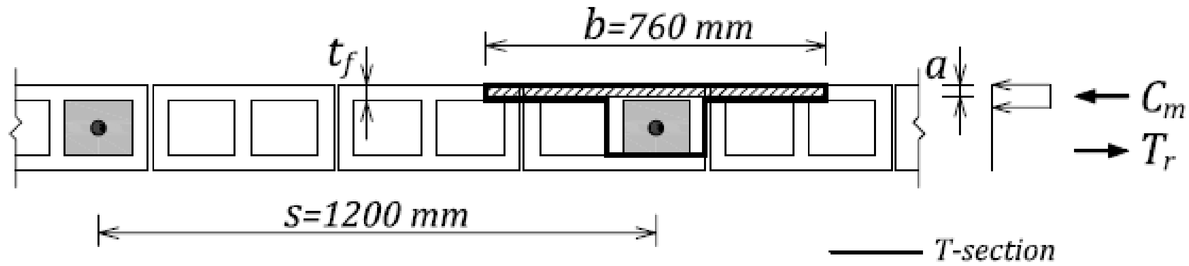
$b = s = 1200$ mm

or

$b = 4t = 4*190 = 760$ mm

Therefore, $b = 760$ mm will be used as the width of the masonry compression zone.

A typical wall cross-section is shown on the figure below. Note that the face shell thickness is 38 mm (typical for a hollow block masonry unit). The same value can be obtained from Table D-1, considering the case of an ungrouted 200 mm block wall.



Since the reinforcement is placed at the centre of the wall, the effective depth is equal to

$$d = \frac{t}{2} = \frac{190}{2} = 95 \text{ mm}$$

The reinforcement area used for the design needs to be determined as follows:

$$A_s = A_b = 200 \text{ mm}^2$$

The internal forces will be determined as follows (see Figure C-8):

$$T_r = \phi_s f_y A_s = 0.85*400*200 = 68000 \text{ N}$$

Since

$$C_m = P_f + T_r = 9700 + 68000 = 77700 \text{ N}$$

and

$$C_m = (0.85\phi_m f'_m)(b \cdot a)$$

the depth of the compression stress block a can be determined as follows

$$a = \frac{C_m}{0.85\phi_m f'_m b} = \frac{77700}{0.85*0.6*9.8*760} = 20 \text{ mm}$$

Since

$$a = 20mm < t_f = 38mm$$

the neutral axis is located in the face shell (flange). The moment resistance around the centroid of the wall section can be determined as follows

$$M_r = C_m(d - a/2) = 77700*(95 - 20/2) = 6.6 \text{ kNm}$$

Since

$$M_r = 6.6 \text{ kNm} = M_f = 6.6 \text{ kNm}$$

it follows that the wall flexural resistance is adequate. However, the reinforcement spacing could be reduced to s =1000 mm to allow for an additional safety margin (the revised moment resistance calculations are omitted from this example).

4. Check whether the out-of-plane shear resistance of the wall is adequate (see Section 2.4.2).

According to S304.1 Cl.10.10.2, the factored out-of-plane shear resistance (V_r) shall be taken as follows

$$V_r = \phi_m(v_m \cdot b \cdot d + 0.25P_d)$$ where

$v_m = 0.16\sqrt{f'_m}$ = 0.50 MPa

$d = 95$ mm effective depth

$b \approx 200$ mm web width - equal to the grouted cell width (156 mm) plus the thickness of the adjacent webs (26 mm each)

The axial load P_d can be determined as

$P_d = 0.9P_f = 0.9 * 9.7 = 8.7$ kN

Thus,

$V_r = 0.6 * (0.50 * 200 * 95 + 0.25 * 8700) = 7.0$ kN

Since

$V_f = 4.0$ kN < $V_r = 7.0$ kN OK

Maximum shear allowed on the section is

$\max V_r = 0.4\phi_m \sqrt{f'_m}(b * d) = 0.4 * 0.6 * \sqrt{9.8} * (200 * 95) = 14.3$ kN OK

5. Check the sliding shear resistance (see Section 2.4.3).

The factored in-plane sliding shear resistance V_r is determined according to S304.1 Cl.10.10.4.2, as follows:

μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

A_s = 200 mm^2 area of vertical reinforcement at 1.2 m spacing

$T_y = \phi_s A_s f_y$ = 0.85*200*400 = 68.0 kN

P_d = 8.7 kN

$P_2 = P_d + T_y$ = 8.7+68.0 = 76.7 kN

$V_r = \phi_m \mu P_2$ = 0.6*1.0*76.7 = 46.0 kN

$V_r = 46.0$ kN > $V_f = 4.0$ kN OK

6. Conclusion

It can be concluded that the out-of-plane seismic resistance of this nonloadbearing wall is satisfactory. It should be noted that the flexural resistance governs in this design. The required amount of vertical reinforcement (15M@1200 mm) corresponds to the following area per metre length

$A_s = A_b * \frac{1000}{s} = 167$ mm^2

which is significantly larger than the minimum seismic reinforcement prescribed by CSA S304.1, that is, $A_{stotal} = 95$ mm^2/m. Note that 15M@1200 mm is also the minimum vertical reinforcement that meets the minimum spacing requirements using typical15M bars.

Also, since horizontal reinforcement does not contribute to out-of-plane wall resistance, it was not considered in this example. However, provision of 9 Ga. horizontal ladder reinforcement at 400 mm spacing could be considered to improve the overall seismic performance of the wall.

It should be noted that, in exterior walls the mortar-bedded joints could be significantly affected by the presence of aesthetic joint finishes characterized by deeper grooves (e.g. raked joints); some of the grooves are up to 10 mm deep. The designer should consider this effect in the calculation of the compression zone depth.

<u>EXAMPLE 7</u>: Seismic design of masonry veneer ties

Perform the seismic design for tie connections for a 4.8 m high concrete block veneer wall in a school gymnasium in Montréal, Quebec. The building is founded on rock. The design should be performed to the requirements of NBCC 2005, CSA S304.1-04, and CSA A370-04. Consider the following two types of the veneer backup:
a) Concrete block wall (a rigid backup), and
b) Steel stud wall with 400 mm steel stud spacing (a flexible backup).
c) Evaluate the minimum tie strength requirements for the rigid and flexible backup.

<u>SOLUTION:</u>

This design problem requires the calculation of seismic load V_p for nonstructural elements according to NBCC 2005 Cl.4.1.8.17 (for more details see Section 2.6.5.3). Note that the wind load could govern in a tie design for many site locations in Canada, however wind load calculations were omitted for this seismic design example.

First, seismic design parameters need to be determined as follows:

- Location: Montréal, Quebec
  $S_a(0.2)$ = 0.69 (NBCC 2005 Appendix C, page C-26)
- Foundation factors
  Site Class = B (rock)
  F_a = 0.88 for $S_a(0.2)$ =0.69 and Site Class B (by interpolation from Table 1-10 or NBCC 2005 Table 4.1.8.4.B), since
  F_a = 0.8 for $S_a(0.2) =$ 0.50
  F_a = 0.9 for $S_a(0.2) =$ 0.75
- I_E = 1.3 school (high importance building)

At this point, it would be appropriate to check whether the seismic design of ties is required for this design. According to NBCC 2005 Cl.4.1.8.17.2, seismic design of ties is required when the seismic hazard index $I_E F_a S_a(0.2) \geq 0.35$ (and also for post-disaster buildings in lower seismic regions).
In this case,
$I_E F_a S_a(0.2)$= 1.3*0.88*0.69=0.79 $\geq$ 0.35
Therefore, seismic design is required.

- Find S_p (horizontal force factor for part or portion of a building and its anchorage per NBCC 2005, Table 4.1.8.17, Case 8)

$S_p = C_p A_r A_x / R_p = 1.0 \cdot 1.0 \cdot 3.0/1.5 = 2.0$
where
$A_x = 1 + 2h_x/h_n = 3.0$ for top of wall worst case
Since $0.7 < S_p < 4.0$ O.K.

- W_p = 1.8 kN/m^2 unit weight of the veneer masonry (concrete blocks)

Seismic load V_p can be calculated as follows:
$V_p = 0.3 F_a S_a(0.2) I_E S_p W_p$=0.3*0.88*0.69*1.3*2.0*(1.8 kN/m^2) =0.85 kN/m^2
Note that the above load is determined per m^2 of the wall surface area.

a) Concrete block backup (rigid)
Assume the maximum tie spacing permitted according to CSA S304.1 Cl.9.1.3 of 600 mm vertically and 820 mm horizontally (see Section 2.6.5.2), resulting in a tributary tie area for a concrete backup wall of
A = 0.82*0.60 = 0.49 m^2
The required factored tie capacity should exceed the factored tie load, that is,
$V_f \geq V_p * A$ = (0.85 kN/m^2)*(0.49 m^2) = 0.42 kN
Alternatively, for a given tie capacity, a tie spacing could be determined based on the maximum tributary area calculated from V_p and the factored tie capacity V_f , that is,
$A \leq V_f / V_p$

b) Steel stud backup (flexible)
Since the steel stud is a flexible backup, a tie must be able to resist 40% of the tributary lateral load on a vertical line of ties (S304.1 Cl.9.1.3.3, see Section 2.6.5.3):
$V_f \geq 0.4 * V_p * A_t$ = 0.4*(0.85 kN/m^2)*(1.92m^2) = 0.65 kN
where A_t = 0.4m*4.8m = 1.92 m^2 is tributary area on a vertical line of ties based on a probable 0.4 m horizontal tie spacing, and 4.8 m wall height

According to the same S304.1 clause, the tie must also be able to resist a load corresponding to double the tributary area on a tie, that is,
$V_f = 2 * V_p * A$ = 2*(0.85 kN/m^2)*(0.4m*0.6m) = 0.41 kN
Note that the tributary area was based on a 0.4 m stud spacing, and the maximum vertical tie spacing of 0.6 m prescribed by S304.1 Cl. 9.1.3.1.

In conclusion, the tie design load for the flexible veneer backup is V_f = 0.65 kN.

c) Minimum strength requirements
CSA A370-04 Cl.8.1 prescribes minimum ultimate tensile/compressive tie strength of 1 kN. In order to obtain the ultimate tie strength, the factored strength needs to be divided by the resistance factor ϕ. According to CSA A370-04 Cl.9.4.2.1.2, the resistance factor is 0.9 for tie material strength, or 0.6 for embedment failure, failure of fasteners, or buckling failure of the connection. It is conservative to use lower resistance factor in determining the ultimate tie strength V_{ult}.

- For the steel stud backup:

$V_r \geq V_f = 0.65$ kN
thus the ultimate strength can be determined as follows
$$V_{ult} = \frac{V_r}{\phi} = \frac{0.65}{0.6} = 1.08 \text{ kN}$$
This value is slightly higher than the minimum of 1 kN prescribed by CSA A370-04 and governs.

- For the concrete block backup:

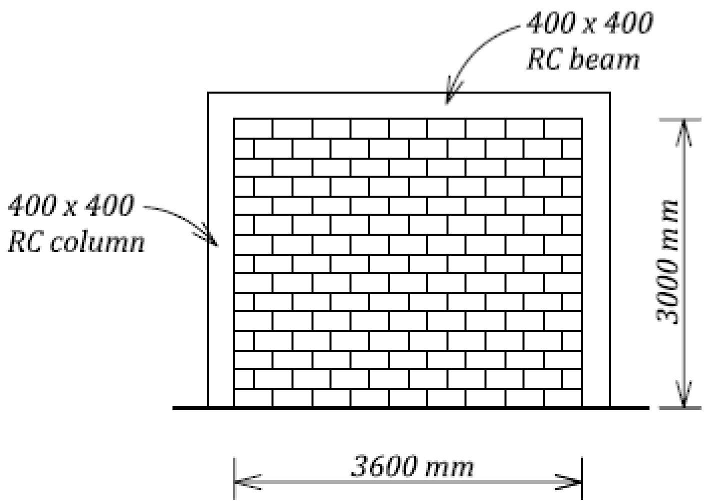
$V_r \geq V_f = 0.42$ kN
thus the ultimate strength can be determined as follows
$$V_{ult} = \frac{V_r}{\phi} = \frac{0.42}{0.6} = 0.7 \text{ kN}$$
This value is less than the minimum of 1 kN, so the minimum requirement governs.

EXAMPLE 8: Seismic design of a masonry infill wall

A single-storey reinforced concrete frame structure is shown in the figure below. The frame is infilled with an unreinforced, ungrouted concrete block wall panel that is in full contact with the frame. The wall is built using 190 mm hollow blocks and Type S mortar.

a) Model the infill as an equivalent diagonal compression strut. Determine the strut dimensions according to CSA S304.1 assuming the infill-frame interaction.

b) Assuming that the infill wall provides the total lateral resistance, determine the maximum lateral load that the infilled frame can resist. Consider the following three failure mechanisms: strut compression failure, diagonal tension resistance, and sliding shear resistance.



Given:

E_f =25000 MPa concrete frame modulus of elasticity

f'_m = 9.8 MPa hollow block masonry, from 15 MPa block strength and Type S mortar (Table 4, CSA S304.1)

SOLUTION:

a) Find the diagonal strut properties.

- Key properties for the masonry wall and the concrete frame

Concrete frame:

E_f =25000 MPa

Beam and column properties:

$$I_b = I_c = \frac{(400)^4}{12} = 2.133*10^9 \text{ mm}^4$$

Masonry:

$E_m = 850 f'_m = 850 * 9.8 = 8330$ MPa

Effective wall thickness (face shells only):

$t_e = 75$ mm (Table D-1, 200 mm hollow block wall)

- Diagonal strut geometry (see Section 2.6.2 and S304.1 Cl.7.13)

$h = 3000$ mm

$l = 3600$ mm

Find θ (angle of diagonal strut measured from the horizontal):

$$\tan(\theta) = \frac{h}{l} = \frac{3000}{3600} = 0.833 \qquad \theta = 39.8^\circ$$

Length of the diagonal strut:

$$l_s = \sqrt{h^2 + l^2} = \sqrt{3000^2 + 3600^2} = 4686 \text{ mm}$$

Find the strut width (see Figure 2-36):

$$\alpha_h = \frac{\pi}{2}\left(\frac{4E_f I_c h}{E_m t_e \sin 2\theta}\right)^{1/4} = \frac{\pi}{2}\left(\frac{4*25000*2.133*10^9*3000}{8330*75*\sin(2*39.8^\circ)}\right)^{1/4} = 1587$$

$$\alpha_L = \pi\left(\frac{4E_f I_b l}{E_m t_e \sin 2\theta}\right)^{1/4} = \pi\left(\frac{4*25000*2.133*10^9*3600}{8330*75*\sin(2*39.8^\circ)}\right)^{1/4} = 3322$$

Strut width:

$$w = \sqrt{\alpha_h{}^2 + \alpha_L{}^2} = \sqrt{(1587)^2 + (3322)^2} = 3682 \text{ mm}$$

Effective diagonal strut width w_e for the compressive resistance calculation should be taken as the least of (Cl.7.13.3.3)

$w_e = w/2 = 3682/2 = 1841$ mm

or

$w_e = l_s/4 = 4686/4 = 1172$ mm

thus

$w_e = 1172 \approx 1170$ mm

The design length of the diagonal strut l_d should be equal to (Cl.7.13.3.5)

$l_d = l_s - w/2 = 4686 - 3682/2 = 2845$ mm

b) Determine the maximum lateral load which the infilled frame can resist assuming that the infill wall provides the total lateral resistance.

- Diagonal strut: compression resistance (Cl.7.13.3.4 and Section 2.6.2)

The compression strength of the diagonal strut $P_{r\max}$ is equal to the compression strength of masonry times the effective cross-sectional area, that is,

$$P_{r\max} = (0.85\chi\phi_m f'_m)\cdot A_e$$

where

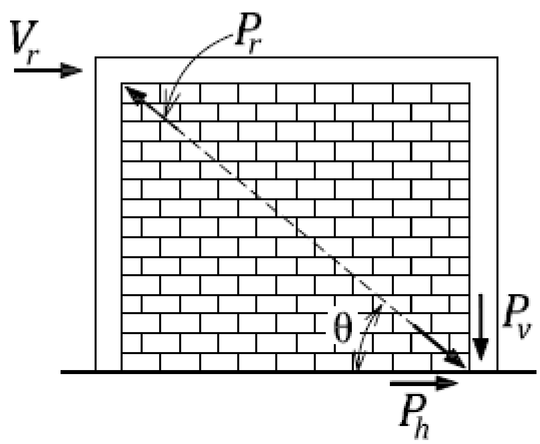
ϕ_m = 0.6

$\chi = 0.5$ the masonry compressive strength parallel to bed joints

$A_e = t_e * w_e = 75*1170 = 87750$ mm^2 the effective cross-sectional area

$P_{r\max} = 0.85*0.5*0.6*9.8*87750 = 219.3$ kN

The corresponding lateral force is equal to the horizontal component of the strut compression force P_h, that is, (see the figure below)
$P_h = P_{r\max} * \cos(\theta) = 219.3*\cos(39.8) = 168.0$ kN



Before proceeding with the design, slenderness effects should also be checked. First, the slenderness ratio needs to be determined as follows (Cl.7.7.5):

$$\frac{k*l_d}{t} = \frac{1.0*2845}{190} = 15.0$$

where
$k = 1.0$ assume pin-pin support conditions
$l_d = 2845$ mm design length for the diagonal strut
$t = 190$ mm overall wall thickness
The strut is concentrically loaded, but the minimum eccentricity needs to be taken into account, that is,
$e_1 = e_2 = 0.1*t = 19$ mm
Since

$$\frac{k*l_d}{t} = 15.0 > 10 - 3.5 e_1/e_2 = 6.5 \text{ and } \frac{k*l_d}{t} < 30.0$$

the slenderness effects need to be considered.

The critical axial compressive force for the diagonal strut P_{cr} will be determined according to S304.1 Cl.7.7.6.3 as follows:

$$P_{cr} = \frac{\pi^2 \phi_{er} E_m I_{eff}}{(1+0.5\beta_d)(kl_d)^2} = 1380 \text{ kN}$$

where
$\phi_{er} = 0.65$
$\beta_d = 0$ assume 100% seismic live load
$E_m = 8330$ MPa modulus of elasticity for masonry
$I_{eff} = 0.4 I_o = 209*10^6$ mm^4
where

$I_o = \frac{1170*\left[190^3-(190-75.4)^3\right]}{12} = 522*10^6$ mm^4 moment of inertia of the effective cross-sectional area based on the effective diagonal strut width $w_e = 1170$ mm and the effective wall thickness $t_e = 75.4$ mm (face shells only).

Since
$P_{r\,max} = 219.3$ kN < $P_{cr} = 1380$ kN
it follows that compression failure governs over buckling failure.

- The diagonal tension shear resistance (see Section 2.3.2 and CSA S304.1 Cl.10.10.1).

Find the masonry shear resistance (V_m):

$b_w = 190$ mm overall wall thickness

$d_v \approx 0.8 l_w = 2880$ mm effective wall depth

$\gamma_g = 0.5$ ungrouted wall

$P_d = 0$ (ignore self-weight)

$v_m = 0.16\sqrt{f'_m}$ = 0.5 MPa

$V_m = \phi_m (v_m b_w d_v + 0.25 P_d)\gamma_g$ = 0.6(0.5*190*2880+0)*0.5 ≈ 82.0 kN

This is a squat shear wall because $\frac{h_w}{l_w} = \frac{3000}{3600} = 0.83 < 1.0$. In this case, there is no need to find the maximum permitted shear resistance per S304.1 Cl.10.10.1.3 $\max V_r$ because it is not going to control for an unreinforced wall without gravity load.

- Sliding shear resistance (see Section 2.6.1 and Cl.7.10.4)

$V_{rs} = 0.16\phi_m\sqrt{f'_m}A_{uc} + \phi_m \mu P_1$

The factored in-plane sliding shear resistance V_r is determined as follows.

μ = 1.0 for a masonry-to-masonry or masonry-to-roughened concrete sliding plane

$A_{uc} = t_e \cdot d_v = 75*2880 = 216000$ mm^2 uncracked portion of the effective wall cross-sectional area

The compressive force in masonry acting normal to the sliding plane is normally taken as P_d plus an additional component, equal to 90% of the factored vertical component of the compressive force resulting from the diagonal strut action P_v (see the figure on the previous page).

$P_1 = P_d + 0.9*P_v$

where

$P_v = V_{rs}*\tan(\theta)$

thus

$P_1 = 0 + 0.9*V_{rs}\tan(\theta)$

The sliding shear resistance can be determined from the following equation

$V_{rs} = 0.16\phi_m\sqrt{f'_m}A_{uc} + \phi_m\mu(0.9*V_{rs}\tan(\theta))$

or

$$V_{rs} = \frac{0.16\phi_m \sqrt{f'_m} A_{uc}}{1-\phi_m * \mu * 0.9 * \tan(\theta)} = \frac{0.16 * 0.6 * \sqrt{9.8} * 216000}{1-0.6*1.0*0.9*\tan(39.8^\circ)} = 118.0 \text{ kN}$$

- Discussion

It is important to consider all possible behaviour modes and identify the one that governs in this design. The following three lateral forces should be considered:

a) $P_h = 168$ kN shear force corresponding to the strut compression failure

b) $V_m = 82$ kN diagonal tension shear resistance

c) $V_{rs} = 118$ kN sliding shear resistance

It could be concluded that the diagonal tension shear resistance governs, however once diagonal tension cracking takes place, the strut mechanism forms. Therefore, the maximum shear force developed in an infill wall corresponds either to the strut compression resistance or the sliding shear resistance (see the discussion in Section 2.6.2). In this case, sliding shear resistance governs and so $V_{r\max} = V_{rs} = 118$ kN.

It should be noted that the maximum shear force developed in the infill $V_{r\max}$ will be transferred to the adjacent reinforced concrete columns, which need to be designed for shear. This is not the scope of the masonry design, however the designer should always consider the entire lateral load path and the force transfer between the structural components.

References

Abrams,D.P. (2000). A Set of Classnotes for a Course in: Masonry Structures, Third Edition, The Masonry Society, Boulder, CO, USA.

Abrams,D.P., Angel,R., and Uzarski,J. (1996). Out-of-Plane Strength of Unreinforced Masonry Infill Panels, Earthquake Spectra, Vol.12, No.4, November 1996, pp.825-844.

ACI 530-08 (2008). Building Code Requirements & Specifications for Masonry Structures and Related Commentaries (TMS 402-08/ACI 530-08/ASCE 5-08), American Concrete Institute, Farmington Hills, MI.

Adams, J., and Atkinson,G. (2003). Development of Seismic Hazard Maps for the Proposed 2005 Edition of the National Building Code of Canada, Canadian Journal of Civil Engineering, Vol.30, p. 255-271.

Adams, J., and Halchuk, S. (2003). Fourth Generation Seismic Hazard Maps of Canada: Values for Over 650 Canadian Localities Intended for the 2005 National Building Code of Canada. Geological Survey of Canada, Open File 4459. 155 p. (free download available at http://earthquakescanada.nrcan.gc.ca/hazard/OF4459/index_e.php)

Anderson,D.L. (2006). Dynamic Analysis, Lecture Notes, Understanding Seismic Load Provisions for Buildings in the National Building Code of Canada 2005, Vancouver Structural Engineers Group Society, Vancouver, Canada.

Anderson, D.L. (2006a). CSA S304.1 and NBCC Seismic Design Provisions for Masonry Structures, Lecture Notes, Course E1 Masonry Design of Buildings, Certificate Program in Structural Engineering, Vancouver Structural Engineers Group Society and the UBC Department of Civil Engineering, Vancouver, Canada (unpublished).

Anderson, D.L., and Priestley, M.J.N. (1992). In Plane Shear Strength of Masonry Walls, Proceedings, The Sixth Canadian Masonry Symposium, Department of Civil Engineering, University of Saskatchewan, Saskatoon, Saskatchewan, Canada, Vol.2, p.223-234.

Amrhein,J.E., Anderson,J.C., and Robles,V.M. (1985). Mexico Earthquakes - September 1985, The Masonry Society Journal, Vol.4, No.2, July-December 1985, pp.G12-G17.

Bachmann,H. (2003). Seismic Conceptual Design of Buildings – Basic Principles for Engineers, Architects, Building Owners, and Authorities, Swiss Federal Office for Water and Geology, Swiss Agency for Development and Cooperation, Switzerland. (free download available at http://www.preventionweb.net/english/professional/publications/v.php?id=687)

Bruneau,M., Uang,C.M., and Whittaker,A. (1998). Ductile Design of Steel Structures, McGraw-Hill, New York, p. 485.

Brzev, S. (2006). Seismic Design of Masonry Shear Walls, Lecture Notes, Course E1 Masonry Design of Buildings, Certificate Program in Structural Engineering, Vancouver Structural Engineers Group Society and the UBC Department of Civil Engineering, Vancouver, Canada (unpublished).

Cardenas, A.E., and Magura, D.D. (1973). Strength of High-Rise Shear Walls — Rectangular Cross Section, Response of Multistory Concrete Structures to Lateral Forces, ACI Publication SP-36, American Concrete Institute, Detroit, pp. 119–150.

Chopra, A.K. (2007). Dynamics of Structures: Theory and Applications to Earthquake Engineering, 3rd Edition, Prentice Hall Inc., Upper Saddle River, NJ, USA.

Corley, W.G. (1966). Rotational Capacity of Reinforced Concrete Beams. Journal of the Structural Division, ASCE, Vol.92, No.ST10, 121-146.

CSA A23.3-04 (2004). Design of Concrete Structures, Canadian Standards Association, Mississauga, Ontario, 233 pp.

CSA A370-04 (2004). Connectors for Masonry, Canadian Standards Association, Mississauga, Ontario.

CSA A371-04 (2004). Masonry Construction for Buildings, Canadian Standards Association, Mississauga, Ontario.

CSA S304.1-04 (2004). Design of Masonry Structures, Canadian Standards Association, Mississauga, Ontario.

CSA S304.1-94 (1994). Masonry Design for Buildings (Limit States Design), Canadian Standards Association, Etobicoke, Ontario.

Dawe,J.L., and Seah,C.K. (1989). Out-of-Plane Resistance of Concrete Masonry Infilled Panels, Canadian Journal of Civil Engineering, Ottawa, Ontario, Vol.16, pp. 854-864.

DeVall, R. (2003). Background Information for Some of the Proposed Earthquake Design Provisions for the 2005 edition of the National Building Code of Canada. Canadian Journal of Civil Engineering, Vol.30, p. 279-286.

Drysdale, R.G., and Hamid, A.A. (2005). Masonry Structures: Behaviour and Design, Canadian Edition, Canada Masonry Design Centre, Mississauga, Ontario.

Elshafie,H., Hamid,A., and Nasr, E. (2002). Strength and Stiffness of Masonry Shear Walls with Openings, The Masonry Society Journal, Vol.20, No.1, December 2002, pp.49-60.

FEMA 306 (1999). Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings- Basic Procedures Manual (FEMA 306). Federal Emergency Management Agency, Washington, D.C., USA.

FEMA 307 (1999). Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings-Technical Resources (FEMA 307). Federal Emergency Management Agency, Washington, D.C., USA.

FEMA 99 (1995). A Nontechnical Explanation of the 1994 NEHRP Recommended Provisions, Federal Emergency Management Agency, Washington, D.C., USA.

Ferguson, P.M., Breen,J.E., and Jirsa.,J.O. (1988). Reinforced Concrete Fundamentals, 5th Edition, John Wiley&Sons, New York, USA.

Hatzinikolas, M.A., and Korany, Y. (2005). Masonry Design for Engineers and Architects, Third Edition, Canadian Masonry Publications, Edmonton, AB, Canada.

Henderson, R.C., Bennett, R., and Tucker, C.J. (2007). Development of Code-Appropriate Methods for Predicting the Capacity of Masonry Infilled Frames Subjected to In-Plane Forces, Final Report Submitted to the National Concrete Masonry Association, USA.

Ingham,J.M., Davidson,B.J., Brammer,D.R., and Voon, K.C. (2001). Testing and Codification of Partially Grout-Filled Nominally Reinforced Concrete Masonry Subjected to In-Plane Cyclic Loads, The Masonry Society Journal, Vol.19,No.1, September 2001, pp.83-96.

Kaushik,H.B., Rai,D.C., and Jain,S.K. (2006). Code Approaches to Seismic Design of Masonry-Infilled Reinforced Concrete Frames: A State-of-the-Art Review, Earthquake Spectra, Earthquake Engineering Research Institute, Vol.22, No.4, pp.961-983.

Klingner,R.E. (2005). Masonry Course Notes, The University of Texas at Austin, The Masonry Society, Boulder, CO, USA.

Leiva,G., and Klingner,R.E. (1994). Behavior and Design of Multi-Story Masonry Walls Under In-Plane Seismic Loading, The Masonry Society Journal, Vol.13, No.1, August 1994, pp.15-24.

Leiva,G., Merryman,M., and Klingner,R.E. (1990). Design Philosophies For Two-Story Concrete Masonry Walls with Door and Window Openings, Proceedings of the Fifth North American Masonry Conference, University of Illinois at Urbana-Champaign, June 1990, pp.287-295.

Matsumura, A. (1987), "Shear Strength of Reinforced Hollow Unit Masonry Walls", Proceedings of the 4th North American Masonry Conference, Paper No. 50, Los Angeles, USA.

MacGregor, J.G., and Bartlett, F.M. (2000). Reinforced Concrete – Mechanics and Design, First Canadian Edition, Prentice Hall Canada Inc., Scarborough, Ontario.

MIBC (2008). Masonry Technical Manual, Masonry Institute of British Columbia, 140 pp. (free download available at www.masonrybc.org)

Mitchell, D., et al. (2003). Seismic force modification factors for the proposed 2005 edition of the National Building Code of Canada. Canadian Journal of Civil Engineering, Vol.30, p. 308-327.

Murty, C.V.R, Brzev, S., Faison, H., Comartin, C.D., and Irfanoglu, A. (2006). At Risk: The Seismic Performance of Reinforced Concrete Frame Buildings with Masonry Infill Walls, Earthquake Engineering Research Institute, Publication No. WHE-2006-03, First Edition, 70 pp. (free download available at www.world-housing.net)

Murty, C.V.R. (2005). IITK-BMPTC Earthquake Tips – Learning Earthquake Design and Construction. National Information Center of Earthquake Engineering, IIT Kanpur, India. (free download available at http://www.nicee.org/EQTips.php)

Naeim, F. (2001). The Seismic Design Handbook, 2nd Edition, Kluwer Academic Publisher, USA.

Nathan, N.D. Philosophy of Seismic Design, Department of Civil Engineering, University of British Columbia, Vancouver, Canada, pp.93.

NRC (2006). User's Guide – NBC 2005 Structural Commentaries (Part 4 of Division B), Canadian Commission on Building and Fire Codes, National Research Council Canada, Ottawa.

NRC (2005). National Building Code of Canada 2005, National Research Council, Ottawa.

NRC (1996). User's Guide – NBC 1995 Structural Commentaries (Part 4), Canadian Commission on Building and Fire Codes, National Research Council Canada, Ottawa.

NRC (1995). National Building Code of Canada 1995, National Research Council, Ottawa.

NZCMA (2004). User's Guide to NZS 4230:2004 Design of Reinforced Concrete Masonry Structures, New Zealand Concrete Masonry Association Inc., Wellington, New Zealand, pp. 83 (http://www.cca.org.nz/shop/downloads/NZS4230UserGuide.pdf).

NZS 4230:2004, "Design of Reinforced Concrete Masonry Structures", Standards Association of New Zealand, Wellington.

Okamoto, S., et al. (1987), "Seismic Capacity of Reinforced Masonry Walls and Beams", Proceedings of the 18th Joint Meeting of the U.S.-Japan Cooperative Program in Natural Resource Panel on Wind and Seismic Effects, NBSIR 87-3540, National Institute of Standards and Technology, Gaithersburg, pp. 307-319.

Park, R. and Paulay, T. (1975). Reinforced Concrete Structures. John Wiley & Sons, Inc, pp. 769.

Paulay, T. (1986) The Design of Ductile Reinforced Concrete Structural Walls for Earthquake Resistance, Earthquake Spectra, Vol.2, No.4, 1986, p. 783-823.

Paulay, T. and Priestley, M.J.N. (1992). Seismic Design of Concrete and Masonry Buildings, John Wiley and Sons, Inc., New York, USA, pp.744.

Paulay, T. and Priestley, M.J.N. (1993). Stability of Ductile Structural Walls, ACI Structural Journal, Vol.90, No.4, pp.385-392.

Priestley, M.J.N., Verma,R, and Xiao,Y. (1994). Seismic Shear Strength of Reinforced Concrete Columns, ASCE, Journal of Structural Engineering, Vol. 120, No.8, p.2310-2329.

Priestley, M.J.N. and Limin, H. (1990). Seismic Response of T-Section Masonry Shear Walls, Proceedings of the Fifth North American Masonry Conference, University of Illinois at Urbana-Champaign, June 1990, p.359-372.

Priestley,M.J.N., and Hart,G. (1989). Design Recommendations for the Period of Vibration of Masonry Wall Buildings, Structural Systems Research Project, Department of Applied Mechanics and Engineering Sciences, Univeristy of California, San Diego and Department of Civil Engineering, University of California, Los Angeles, Report SSRP-89/05, 46 pp.

Priestley, M.J.N. (1982). Ductility of Confined and Unconfined Concrete Masonry Shear Walls, The Masonry Society Journal, Vol.1, No.2, July-December 1982.

Schultz, A.E. (1996), "Seismic Performance of Partially-Grouted Masonry Shear Walls", Proceedings of the 11th World Conference on Earthquake Engineering, CD-Rom Paper No. 1221, Acapulco, Mexico, pp. 246-256.

Shing, P.B., Schuller, M., Klamerus,E., Hoskere,V.S., and Noland,J.L. (1989). Design and Analysis of Reinforced Masonry Shear Walls, Proceedings, The Fifth Canadian Masonry Symposium, Department of Civil Engineering, University of British Columbia, Vancouver, BC, Canada, Vol.2, pp.291-300.

Shing, P.B., Schuller, M., and Hoskere, V. S. (1990), "In-Plane Resistance of Reinforced Masonry Shear Walls", ASCE Journal of Structural Engineering, Vol. 116, No. 3, pp. 619-640.

Shing, P.B. et al. (1990a). Flexural and Shear Response of Reinforced Masonry Walls, ACI Structural Journal, Vol.87, No.6, p.646-656.

Shing, P.B., Schuller, M., and Hoskere, V. S. (1990b), Strength and Ductility of Reinforced Masonry Shear Walls, Proceedings of the 5th North America Masonry Conference, University of Illinois, Urbana-Champaign, pp. 309-320.

Stafford Smith, B. and Coull, A., (1991). Tall Building Structures: Analysis and Design, John Wiley&Sons, Inc., Canada, 537 pp.

Stafford-Smith, B. (1966). Behaviour of Square Infilled Frames, Journal of the Structural Division, Proceedings of ASCE, Vol.92, No.ST1, pp.381-403.

TMS (1994). Performance of Masonry Structures in the Northridge, California Earthquake of january 17, 1994. The Masonry Society, Boulder, Colorado, pp.100.

Tomazevic, M. (1999). Earthquake-Resistant Design of Masonry Buildings. Imperial College Press, London, U.K.

Trembley, R., and DeVall, R. (2006). Analysis Requirements and Structural Irregularities NBCC 2005, Lecture Notes, Understanding Seismic Load Provisions for Buildings in the National Building Code of Canada 2005, Vancouver Structural Engineers Group Society, Vancouver, Canada.

Voon, K.C. (2007a). In-Plane Seismic Design of Concrete Masonry Structures, A thesis submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Civil and Environmental Engineering, the University of Auckland, New Zealand.

Voon, K.C., and Ingham,J.M. (2006). Experimental In-Plane Shear Strength Investigation of Reinforced Concrete Masonry Walls, ASCE, Journal of Structural Engineering, Vol. 132, No.3, p.400-408.

Voon,K.C., and Ingham,J.M. (2007). Design Expression for the In-Plane Shear Strength of Reinforced Concrete Masonry, ASCE, Journal of Structural Engineering, Vol. 133, No.5, p.706-713.

Wallace,M.A., Klingner,R.E., and Schuller,M.P. (1998). What TCCMAR Taught Us, Masonry Construction, October 1998, p.523-529.

TABLE OF CONTENTS – APPENDIX A

A Comparison of NBCC 1995 and NBCC 2005 Seismic Provisions

This appendix provides a review of the NBCC 1995 seismic design provisions, and compares the base shear force and bending moments for a shear wall structure for both the 1995 and 2005 codes. It provides a means of assessing the changes in the seismic design provisions in the two codes, and is organized so that the sections in this appendix follow the same order as the sections in Section 1.5 of Chapter 1.

A.1 NBCC 1995 Seismic Hazard

Section 1.5.1, Chapter 1

4.1.9.1.6)

The seismic hazard in NBCC 1995 is given by the product $v \cdot S$, where S is a shape function shown in Figure A-1, and v is the zonal velocity ratio. The product $v \cdot S$ is very much like an acceleration response spectrum, as it provides a measure of hazard for different structural periods. The magnitude of v and the shape of S are based on estimates of the peak ground velocity and peak ground acceleration, for a 10% in 50 year probability of non-exceedance (1/475 per year probability). The v value is based directly on the peak ground velocity, while the shape of the S function is based on the ratio of the peak ground acceleration (expressed in terms of g) to the peak ground velocity (expressed in m/sec). For code purposes, these values are represented by the parameters Z_a and Z_v, which are used to define the seismic zones set out in the 1995 code. Eastern sites located on the Canadian Shield have high Z_a/Z_v ratios, because hard rock transmits high frequency waves more readily than does the soil and fractured rock of Western Canada, which generally has $Z_a/Z_v \leq 1$. The result is that the seismic hazard is dependent on two site parameters with a 1/475 per year probability.

Note that $v \cdot S$ does not represent the true seismic hazard as the long period values have been increased to account for higher mode effects in structures. S decreases as $1/\sqrt{T}$ in the longer period range, while $S_a(T)$ in NBCC 2005, which better represents a true spectrum, decreases much more rapidly (as a function of $1/T$ beyond 2 seconds). The higher mode effects in structures in NBCC 2005 are explicitly accounted for by use of the M_v factor.

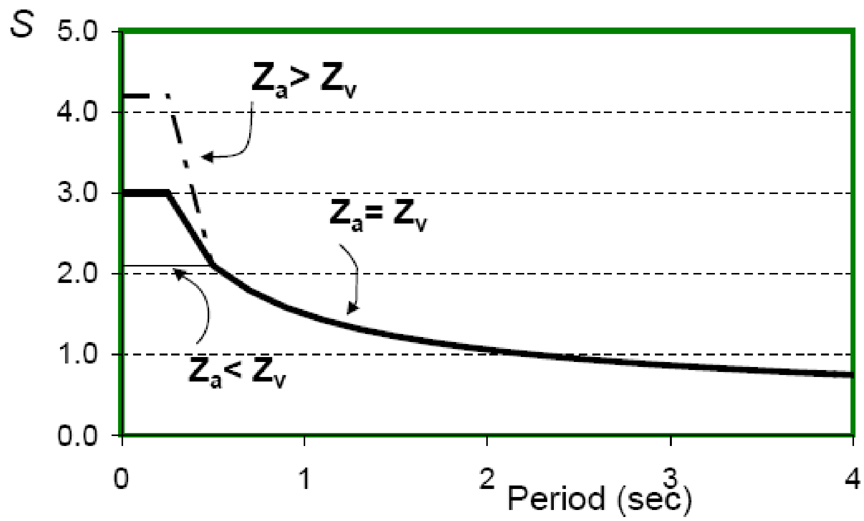


Figure A-1. S function according to NBCC 1995.

A.2 Effect of Site Soil Conditions

Section 1.5.2, Chapter 1

4.1.9.1.11)

The site soil amplification procedure in NBCC 1995 is considerably simpler that that in NBCC 2005. There is only one parameter that multiplies the S function, although there are limits on the amplification in the short period region for some sites.

F denotes the foundation factor which is given in Table A-1. It is applied as a multiplier to S, with the restriction that

$F \cdot S \leq 3.0$ where $Z_a \leq Z_v$, and

$F \cdot S \leq 4.2$ where $Z_a > Z_v$,

i.e., the foundation factor need not increase the short period end of the S function except when $Z_a < Z_v$.

Table A- 1. NBCC 1995 Foundation Factors

Foundation Factors		
Category	***Type and Depth of Soil Measured from the Foundation or Pile Cap Level***	F
1	Rock, dense and very dense coarse-grained soils, very stiff and hard fine-grained soils; compact coarse-grained soils and firm and stiff fine-grained soils from 0 to 15 m deep	1.0
2	Compact coarse-grained soils, firm and stiff fine-grained soils with a depth greater than 15 m; very loose and loose coarse-grained soils and very soft and soft fine-grained soils from 0 to 15 m deep	1.3
3	Very loose and loose coarse-grained soils with depth greater than 15 m	1.5
4	Very soft and fine-grained soils with depth greater than 15 m	2.0

A.3 Methods of Analysis

Section 1.5.3, Chapter 1

4.1.9.1.13.b)

NBCC 1995 does not prescribe a specific method of seismic analysis for building structures. However, Cl.4.1.9.1.13.b) related to vertical force distribution, states that the total lateral seismic force V shall be distributed by means of an equivalent static analysis procedure (part a), or by dynamic analysis with the seismic effects scaled so that the base shear from the dynamic analysis equals V (part b). Commentary J to the NBCC 1995 (NRC, 1996) states that the application of dynamic analysis pertains "especially to buildings with significant irregularities either in plan or elevation, and buildings with setbacks or major discontinuities in stiffness or mass. Performing a dynamic analysis will lead to a better representation of modal contribution in tall buildings."

A.4 Base Shear Calculations

Section 1.5.4, Chapter 1

4.1.9.1.4)

The formula for the design base shear V according to the NBCC 1995 is:

$$V = \left(\frac{V_e}{R}\right)U$$

where

$$V_e = v \cdot S \cdot I \cdot F \cdot W$$

represents the elastic shear force.

The design parameters used in the NBCC 1995 base shear equation are explained in Table A-2. A comparison of V between the 1995 and 2005 codes is presented in Section A.12.

NBCC 1995 (Cl.4.1.9.1.7) prescribes the following relations for the fundamental period T of wall structures:

a) $T = 0.09 h_n \sqrt{D_s}$

where

h_n (m) is building height from the base i.e. top of foundations to the roof level,
D_s (m) is the length of wall or braced frame which constitutes the main lateral load-resisting system in a direction parallel to the applied forces. When the length of the lateral load resisting system is not well defined, then the Code requires that D, the length of building in the direction parallel to the applied forces, shall be used instead of D_s.

b) other established methods of mechanics; with the restriction that the value of V_e used for design shall be not less than 0.80 of the value computed using the period calculated in a).

The period given by the formula (a), which is based on measured values, is a conservative (low) estimate from the data, and generally is smaller than that found using method (b), particularly if the length D is used in the calculation. The code adopted this low estimate as it leads to higher, more conservative, forces. The limit prescribed in (b) is applied to the base shear and not to the period, as the base shear is very sensitive to period in some areas.

Table A- 2. NBCC 1995 Seismic Design Parameters

v =	zonal velocity ratio for the site from the climatic data table in Appendix C of NBCC 1995, based on ground motion associated with a 10% probability of exceedance in 50 years (475 year earthquake).
S =	the seismic response factor, dependent on the Z_a/Z_v ratio for the site and the period T of the structure (see Section A.1).
I =	Importance factor for the structure, equal to I=1.5 for "post-disaster" structures, 1.3 for schools, and 1.0 for ordinary structures;
F =	Foundation factor related to soil conditions (see Section A.2 and Table A-1)
W =	dead weight plus some portion of live load that would move laterally with the structure. Live loads considered are 25% of the snow load, 60% of storage loads for areas used for storage, and the full contents of any tanks. 100% of the live loads are not used as the probability of that occurring at the same time as the earthquake is small. Also, live loads such as people or cars would not move with the same motion as the building.
R =	force modification factor that represents the capability of a structure to dissipate energy through cyclic inelastic (ductile) behaviour. For masonry structures designed and detailed according to CSA S304.1-94: *R* = 2.0 for reinforced walls with nominal ductility, 1.5 for regular reinforced masonry and 1.0 for unreinforced masonry.
U =	0.60, and is described as a "factor representing level of protection based on experience". U was introduced so as to make the design base shear for the 1995 code similar to that in previous codes. Some persons later thought of U as being an overstrength factor, recognizing that the structure has strength higher than the nominal yield strength, but this was not the basis for the introduction of U .

A.5 Force Reduction Factor R

Section 1.5.5, Chapter 1

4.1.9.1.8)

NBCC 1995 had only one R factor, equivalent to the R_d factor in NBCC 2005. NBCC 1995 Table 4.1.9.1.B allows $R = 2$ for reinforced masonry with nominal ductility, $R = 1.5$ for regular reinforced masonry, and $R = 1$ for unreinforced masonry. These values are equivalent to walls with moderate ductility, conventional construction and unreinforced masonry, respectively, in NBCC 2005. Height limitations, and some other provisions that required reinforced masonry, were given in Clause 4.1.9.3 Special Provisions NBCC 1995.

A.6 Higher Mode Effects

Section 1.5.6, Chapter 1

NBCC 1995 does not explicitly mention higher mode effects in calculating the base shear *V*, but the *S* function has been set artificially high in the long period region to account for the contribution from the higher modes. Higher mode effects are considered in the distribution of forces along the height of the structure, see Section A.7, and in calculating the overturning moments, Section A.8.

Note that in contrast to NBCC 2005, the higher mode effects in the 1995 code make no distinction between walls or frames.

A.7 Vertical Distribution of Seismic Forces

Section 1.5.7, Chapter 1

4.1.9.1.13.a)

The distribution of the inertial forces to the floors in NBCC 1995 is essentially the same as in NBCC 2005, and is summarized below

$$F_x = (V - F_t) \cdot \frac{W_x h_x}{\sum_{i=1}^{n} W_i h_i}$$

where
F_x – seismic force acting at level x
W_x - portion of W that is assigned to level x
h_x – height from the base of the structure up to the level x
F_t – a portion of the base shear to be applied as an additional force to F_n at the top of the building, and is given by

$F_t = 0$ for T_a < 0.7 sec

$F_t = 0.07 T_a V$ for 0.7 < T_a < 3.6 sec

$F_t = 0.25V$ for T_a > 3.6 sec

where T_a is the fundamental lateral period.

Once the forces at each floor are established, the total storey shears can simply be calculated using statics.

A.8 Overturning Moments (J factor)

Section 1.5.8, Chapter 1

4.1.9.1.23-27

In NBCC 1995, the overturning moment, M , at the base of the structure, shall be reduced by the factor J , where

$J = 1$ for $T < 0.5s$

$J = 1 - 0.2T$ for $0.5s < T < 1.5s$

$J = 0.8$ for $T > 1.5s$

The overturning moment M_x at any level x shall be multiplied by J_x , where

$$J_x = J + (1 - J)(h_x / h_n)^3$$

where h_n is the height to the top of the structure.

Unlike NBCC 2005, the J factor in NBCC 1995 is not dependent on the structure type or the site conditions.

A.9 Torsion

Section 1.5.9, Chapter 1

4.1.9.1.28)

At each storey level throughout the building, the torsional moment applied is taken as one of the following four cases:

i) $T_x = F_x(1.5e_x + 0.1D_{nx})$

ii) $T_x = F_x(1.5e_x - 0.1D_{nx})$

iii) $T_x = F_x(0.5e_x + 0.1D_{nx})$

iv) $T_x = F_x(0.5e_x - 0.1D_{nx})$

where

F_x is lateral force at the x^{th} floor level,

e_x is the eccentricity at level x, and is distance between the centre of mass and the centre of rigidity in the direction perpendicular to the direction of F_x, and

D_{nx} is a plan dimension of the building at level x perpendicular to the direction of F_x.

Note that $0.1D_{nx}$ is termed the accidental eccentricity.

Each element in the building must be designed for the most severe effect of the above load cases.

Note that it is necessary to explicitly determine the value of e_x. However, if a static 3-D structural analysis program is available, it is possible to use a combination of two analyses to determine $F_x(1.5e_x)$ and $F_x(0.5e_x)$ without explicitly determining the e_x.

Alternately, if a 3-D dynamic analysis is carried out the effects of accidental eccentricity should be accounted for by combining the dynamic analysis element forces with the results from a static analysis of either of the two cases of accidental torques given by:

$T_x = +F_x(0.1D_{nx})$, or

$T_x = -F_x(0.1D_{nx})$

In all of the above analyses, F_x represents the storey force from the static analysis described earlier.

A.10 Irregularities and Restrictions

Section 1.5.10, Chapter 1

4.1.9.3)

NBCC 1995 has very few restrictions regarding irregularities. Masonry is specifically mentioned as requiring reinforcement if Z_a or Z_v is 2 or higher, but there are no height limitations based on irregularities as found in NBCC 2005.

A.11 Displacements

Section 1.5.11, Chapter 1

4.1.9.2.1-3)

In NBCC 1995, displacements are to be calculated using the reduced design forces as given by *V*, and then multiplied by *R* to give realistic values. Since *V* is given by

$$V = \left(\frac{V_e}{R}\right)U \quad ,$$

this would imply that the displacements are the elastic displacements reduced by the factor *U*, which has been a somewhat controversial issue. One difference in the codes, is that V_e in the 1995 code is multiplied by the importance factor *I*, while in NBCC 2005 the displacements are not dependent on the importance factor.

The drift ratio limits in NBCC 1995 are 0*.01 for post-disaster buildings* and *0.02 for all other structures*. This is essentially the same as NBCC 2005, except for ordinary structures which can have a drift ratio of 0.025. Overall, the drift limits in NBCC 2005 are tighter than in the 1995 code.

A.12 Shear and Moment Comparison

This section provides a comparison of the base shear and base moment for ductile masonry walls under the NBCC 1995 and NBCC 2005 codes, for periods ranging from very short to four seconds. For ductile masonry shear walls, Toronto and Vancouver have been selected to investigate the effect of the different spectral shapes between eastern and western Canada.

Figure A-2a shows the shear comparison for a site in Toronto, and Figure A-2b for Vancouver. It is assumed that both sites are on firm ground with no soil amplification (site Class C per NBCC 2005). The following force modification factors were used:
$R=2$ and $U=0.6$ for the NBCC 1995 code calculation, and
$R_d=2$ and $R_o=1.5$ for the NBCC 2005 values

In each plot, the line titled 'NBC 2005 spectral shape' represents the *V/W* ratio for the 2005 code, for the same values of R_d and R_o used in the design calculations, but without considering the upper and lower bounds on *V* per NBCC 2005, and with M_v=1 for all periods.

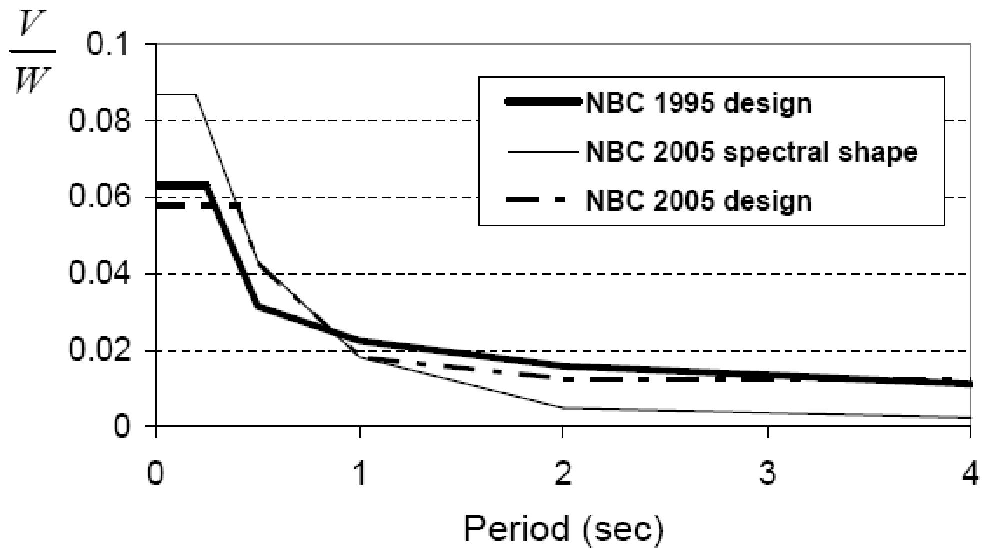
The comparison for Toronto in Figure A-2a shows that there is not much difference in the design level base shear between the codes, with the 2005 code values being lower in the short and long period ranges, but higher for intermediate periods. At a period of 2 seconds, the M_v value is equal to 2.5 for Toronto. The effect of this in increasing the shear is very apparent at the longer periods when compared to the NBCC 2005 spectral shape. Also, it is apparent that without the short period cutoff, the short period shears from NBCC 2005 would be much larger than the NBCC 1995 values.

The comparison for Vancouver in Figure A-2b shows that the NBCC 2005 design base shear is larger than the NBCC 1995 base shear over the entire period range, especially around 0.5 seconds. The M_v factor for Vancouver is equal to 1.2 at the period of 2 seconds, and so has little effect on the long period base shear.

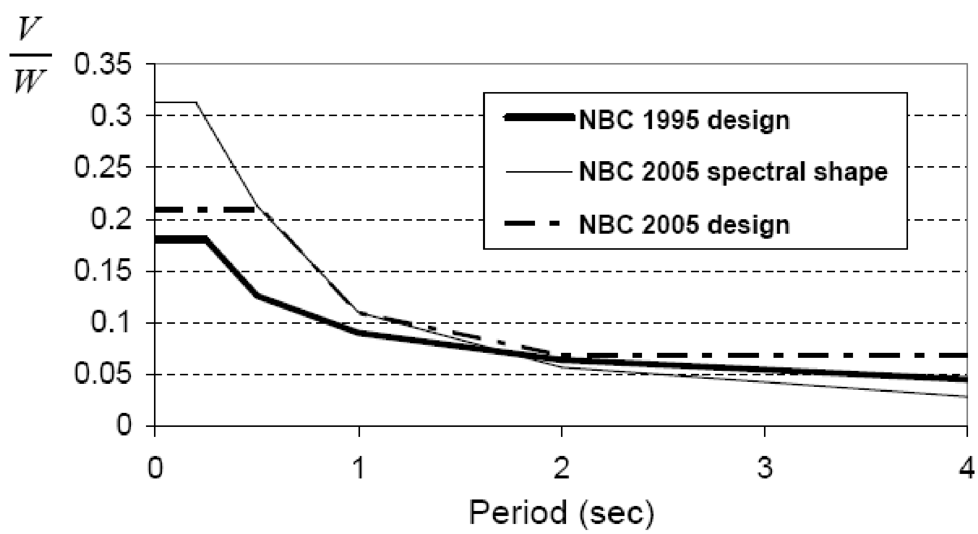
In general, it appears that the base shear from NBCC 2005 is larger than that from NBCC1995. However, because the periods given by the two codes may be different, and because the limit placed on using a longer calculated period is more liberal in the short period end in NBCC 2005, it may be that in some cases there may be a smaller difference in design base shear than the figures indicate.

Since wall size and reinforcement are mainly governed by the wall moments, a moment comparison of the two codes may be more meaningful than a shear comparison.

Figure A-3 compares the base bending moment for NBCC 1995 and NBCC 2005 for the same cases as shown in Figure A-2. The units are not particularly meaningful, but allow a comparison to be made between the two codes. In the short period range less than 1.0 seconds, the moment comparisons are essentially the same as the shear comparisons. But for longer periods, particularly for Toronto, the small value of J at periods of 2.0 s and greater for NBCC 2005 substantially reduces the moments, resulting in much smaller design moments at the longer periods compared to the NBCC 1995 code, as shown in Figure A-3a. For Vancouver, the J factor is larger and does not have as much an effect, but it does bring the design moments from the two codes into close agreement in the longer periods.

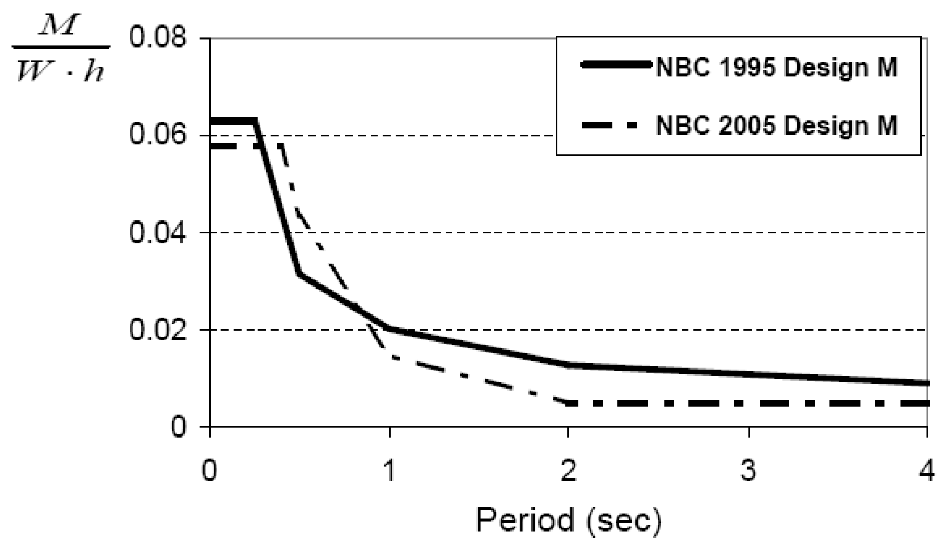


a)

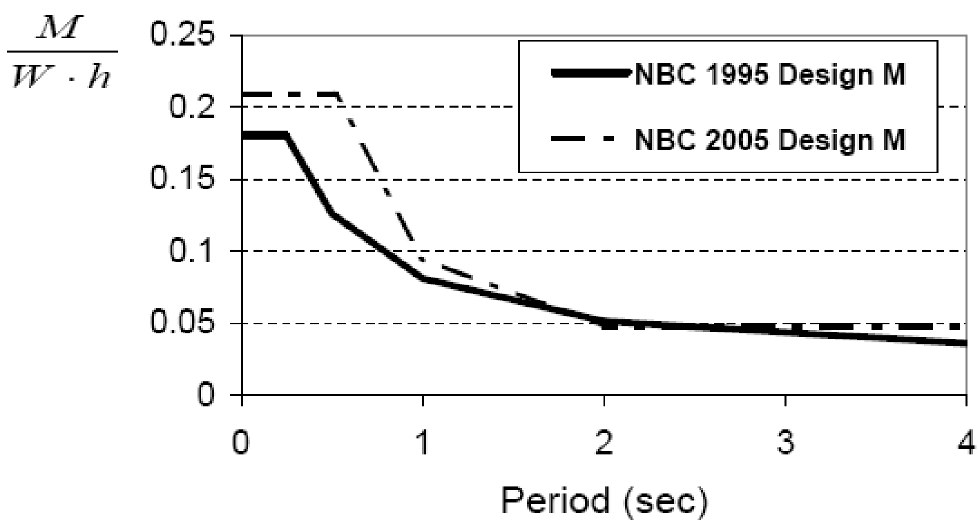


b)

Figure A-2. Base shear comparison for NBCC 1995 and NBCC 2005:
a) Toronto; b) Vancouver.



a)



b)

Figure A-3. Base bending moment comparison for NBCC 1995 and NBCC 2005: a) Toronto; b) Vancouver.

TABLE OF CONTENTS – APPENDIX B

B RESEARCH STUDIES AND CODE BACKGROUND RELEVANT TO MASONRY DESIGN

B Research Studies and Code Background Relevant to Masonry Design

This appendix contains additional background material relevant to the aspects of masonry design discussed in Chapter 2. Findings of some relevant research studies, as well as the discussion on provisions of masonry design codes from other countries, are included. This information may be useful to readers interested in gaining a more detailed insight into the subject. However, it should be noted that designers may use alternative design provisions in situations where CSA S304.1 is silent on a specific issue. The design provisions contained in design standards from other countries cannot supersede the provisions of pertinent Canadian standards.

B.1 Shear/Diagonal Tension Resistance

<u>Axial compression:</u>
An experimental study on reinforced masonry wall specimens by Voon and Ingham (2006) showed that an increase in axial compression stress from 0 to 0.5 MPa resulted in an increase in the maximum wall shear resistance of more than 20%. However, walls subjected to higher axial compression had a reduced post-cracking deformation capacity, resulting in a more brittle failure pattern. The presence of higher axial stress also delayed the onset of diagonal cracking in the walls from the lateral loads, as the vertical stress reduced the principal stress that leads to cracking.

The latest edition of New Zealand Masonry Standard NZS 4230:2004 (SANZ, 2004) prescribes a different method for calculating the axial load contribution to masonry shear resistance than CSA S304.1-04 for low aspect walls. This contribution (equal to $0.9N\tan\alpha$), results from a diagonal strut mechanism, which is based on an assumption that axial compression load N must effectively form a compression strut at an angle α to the axis (see Figure B-1). The axial load must be transmitted through the flexural compression zone, while the horizontal component of the strut force resists the applied shear force (Priestley et al., 1994). This model implies that the shear strength of squat walls under axial loads should be greater than that of more slender walls, and higher than that prescribed in CSA S304.1-04. According to this model, the axial load contribution is limited to $N \le 0.1 f'_m A_g$.

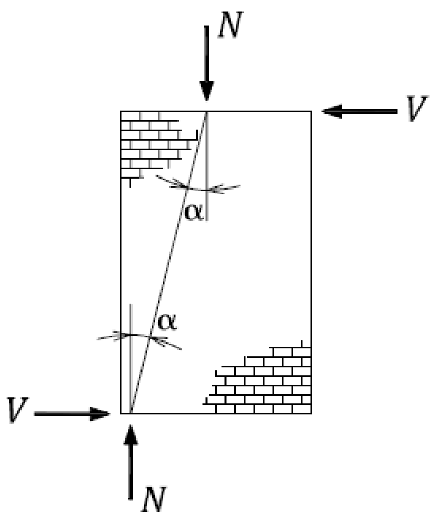


Figure B-1. Contribution of axial load to wall shear strength (reproduced from NZS 4230:2004 with the permission of Standards New Zealand under Licence 000725).

Grouting pattern:

Experimental studies have reported a significant reduction in shear resistance for partially grouted walls compared to otherwise identical fully grouted walls, however partially grouted masonry is a viable lateral load resisting system for regions of low to moderate seismic risk. Schultz (1996) tested a series of six partially grouted reinforced block wall specimens under in-plane cyclic loads. Only the outermost vertical cores and a single course bond beam at midheight were grouted. The mechanism of shear resistance in partially grouted walls is characterized by the development of vertical cracks between ungrouted and grouted masonry due to stress concentrations or planes of weakness (this mechanism is different than the one expected to develop in solidly grouted masonry walls). It was also reported that an increase in horizontal reinforcement ratio did not have a significant effect on the overall shear resistance.

An experimental study by Voon and Ingham (2006) showed that the shear strength of a solidly grouted wall specimen was approximately 110% higher than an otherwise identical specimen with 30% grouted cores. Also, the specimen with 55% grouted cores had more than a 50% higher shear strength compared to the specimen with 30% grouted cores. However, the difference is smaller when the shear stress is compared using the net wall area.

Wall aspect ratio:

The findings of several experimental studies, e.g. Matsumura (1987), Okamoto et al. (1987), and Voon (2007) confirmed that masonry walls with lower aspect ratios exhibited shear strengths that were larger than those for more slender masonry walls. The researchers concluded that the shear strength enhancement was due to the more prominent role of arching action in masonry walls with low aspect ratios, in which shear was mainly resisted by compression struts (see Figure 2-16a).Voon and Ingham (2006) reported that the shear resistance decreased by 15% when the wall aspect ratio increased from 1.0 to 2.0. A squat wall specimen with an aspect ratio of approximately 0.6 showed a significant increase in shear resistance (by over 100%) as compared to a specimen with aspect ratio of 1.0. The findings of an experimental study by Okamoto et al. (1987) confirmed that the wall shear strength increased by 20 to 30% when the aspect ratio decreased from 2.3 to 1.6 and from 2.3 to 0.9 respectively. A study of partially grouted masonry block walls by Schultz (1996) showed that a decrease in the wall aspect ratio was reported to have a beneficial effect on the shear resistance, that is, squat walls are expected to have larger shear resistance than flexural walls of the same height. However, squat wall specimens also showed a reduced deformation capacity and increased strength deterioration.

Steel shear resistance V_s:

Shear reinforcement in masonry walls does not seem to be as effective as in concrete walls. A possible explanation is that the reinforcing bars located where the inclined crack crosses near the end of the bar are unable to develop their full yield strength in the masonry walls. To account for this phenomenon, the New Zealand Masonry Standard NZS 4230:2004 (SANZ, 2004) prescribes a coefficient of 0.8 in the V_s equation, while CSA S304.1-04 uses a 0.6 factor. This phenomenon is particularly pronounced in short walls where it is likely that the length of the shear reinforcement is insufficient to fully develop its yield strength.

It should be acknowledged that horizontal reinforcement in masonry walls usually does not have as good anchorage as the corresponding reinforcement in concrete walls. Anderson and Priestley (1992) have noticed that straight bars or 90° hooks were used in some experimental studies (see Figure B-2a), whereas the horizontal reinforcement in concrete walls is usually anchored in a more effective way, that is, by means of 180° hooks. The type and extent of anchorage are expected to influence the effectiveness of shear reinforcement. Anderson and

Priestley also found that shear strength didn't show any correlation with the vertical reinforcement ratio.

According to some researchers (Shing et al., 1990; Tomazevic, 1999; Voon, 2007), a fraction of the wall shear resistance can be attributed to the presence of vertical reinforcement. Dowel action in vertical reinforcing bars enables shear transfer across a diagonal crack by the localized kinking in reinforcing bars due to their relative displacement (see Figure B-2b) (note that compression kinks cancel out some of the tension kinks). However, once the vertical reinforcement yields, as it would in the plastic hinge zone of ductile walls, its contribution to the shear resistance drops significantly, so CSA S304.1 ignores its contribution to the wall shear resistance.

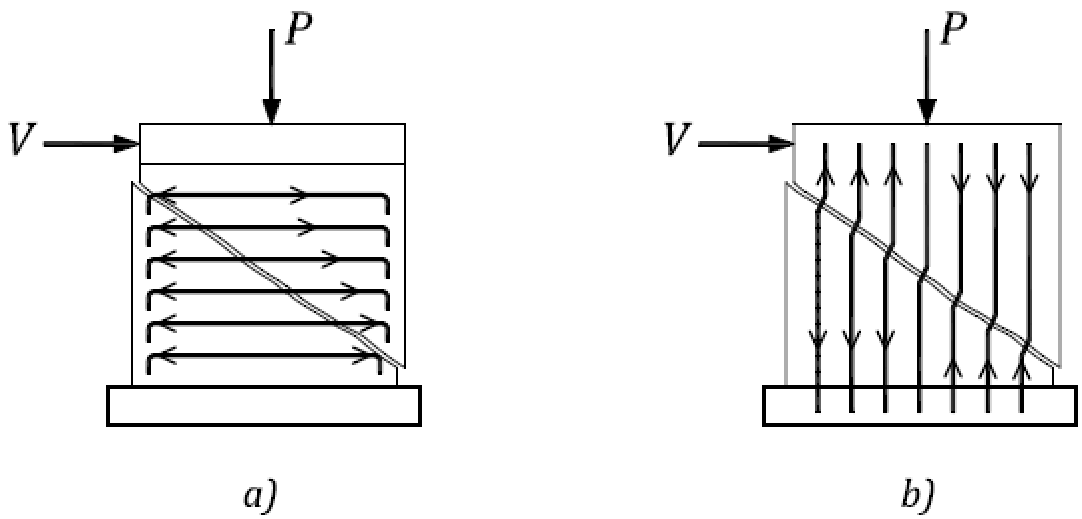


Figure B-2. Wall reinforcement contributing to shear resistance: a) horizontal reinforcement acting in tension; b) dowel action in vertical reinforcement (Tomazevic, 1999, reproduced by permission of the Imperial College Press).

B.2 Ductile Seismic Response

A prime consideration in seismic design is the need to have a structure that is capable of deforming in a ductile manner when subjected to several cycles of lateral loading well into the inelastic range. This section explains a few key terms related to ductile seismic response, including ductility ratio, curvature, plastic hinge, etc. It is very important for a structural designer to have a good understanding of these concepts before proceeding with the seismic design and detailing of ductile masonry walls according to CSA S304.1.

Ductility is a measure of the capacity of a structure or a member to undergo deformation beyond yield level, while maintaining most of its load-carrying capacity. Ductile structural members are able to absorb and dissipate earthquake energy by inelastic (plastic) deformations that are usually associated with permanent structural damage. These inelastic deformations are concentrated mainly in regions called *plastic hinges*. In general, plastic hinges develop in shear walls responding in the flexural mode and are typically formed at their base. An example of a plastic hinge formed in a reinforced masonry wall subjected to seismic loading is shown in Figure 2-8a. The concept of ductility and ductile seismic response was introduced in Section 1.4.3.

A common way to quantify ductility in a structure is through the *displacement ductility ratio* μ_Δ. This is the ratio of the maximum lateral displacement experienced by the structure at the ultimate (Δ_u), to the displacement at the onset of inelastic response (Δ_y) (see Figure 1-5c).

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y}$$

Next, the concept of curvature will be explained by an example of a reinforced masonry shear wall subjected to bending due to a shear force applied at the top, as shown in Figure B-3a. Consider a wall segment ABCD of unit height. This segment deforms due to bending moments, so sections AB and CD rotate by a certain angle relative to their original horizontal position (these deformed sections are denoted as A'B' and C'D'). Rotation between the ends of the segment defines the curvature φ, as shown in Figure B-3b. Curvature represents relative section rotations per unit length. It should be noted that curvature is directly proportional to the bending moment at the wall section under consideration, if the section remains elastic.

Consider any section CD that undergoes curvature φ, as shown in Figure B-3c. Strain distribution along the wall section is defined by the product of curvature and the distance from the neutral axis, located by the depth c. The maximum compressive strain in masonry ε_m is given by

$$\varepsilon_m = \varphi \cdot c$$

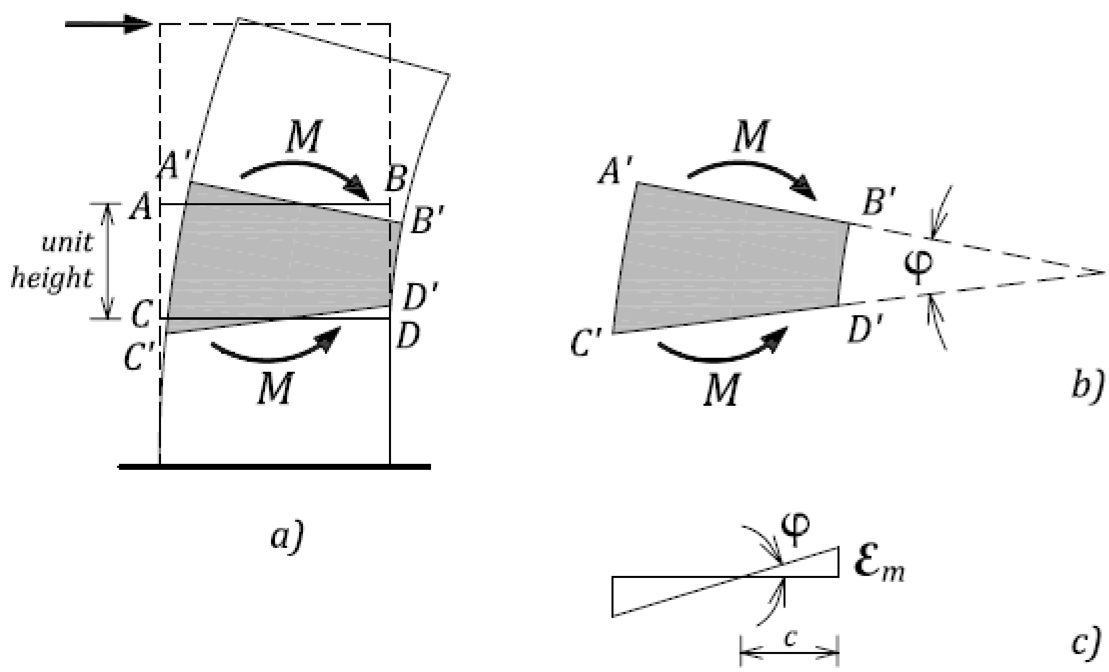


Figure B-3. Curvature in a shear wall subjected to flexure: a) wall elevation; b) deformed wall segment ABCD; c) strain distribution along the section CD.

For the seismic design of reinforced masonry walls, it is of interest to determine curvatures at the following two stages: the onset of steel yielding and at the ultimate stage. Consider a reinforced masonry wall section subjected to axial load and bending shown in Figure B-4a.

Yield curvature φ_y corresponds to the onset of yielding characterized by tensile yield strain ε_y developed in the end rebars, as shown in Figure B-4b, where

$$\varphi_y = \frac{\varepsilon_y}{l_w - d' - c}$$

Ultimate curvature φ_u corresponds to the ultimate stage, when the maximum masonry compressive strain ε_m has been reached. The maximum ε_m value has been limited to 0.0025 by CSA S304.1-04 (see Figure B-4c) to prevent damage to the outer blocks in the plastic hinge region. Note that the neutral axis depth c is going to decrease as more of the reinforcement has yielded (see Figure B-4c).

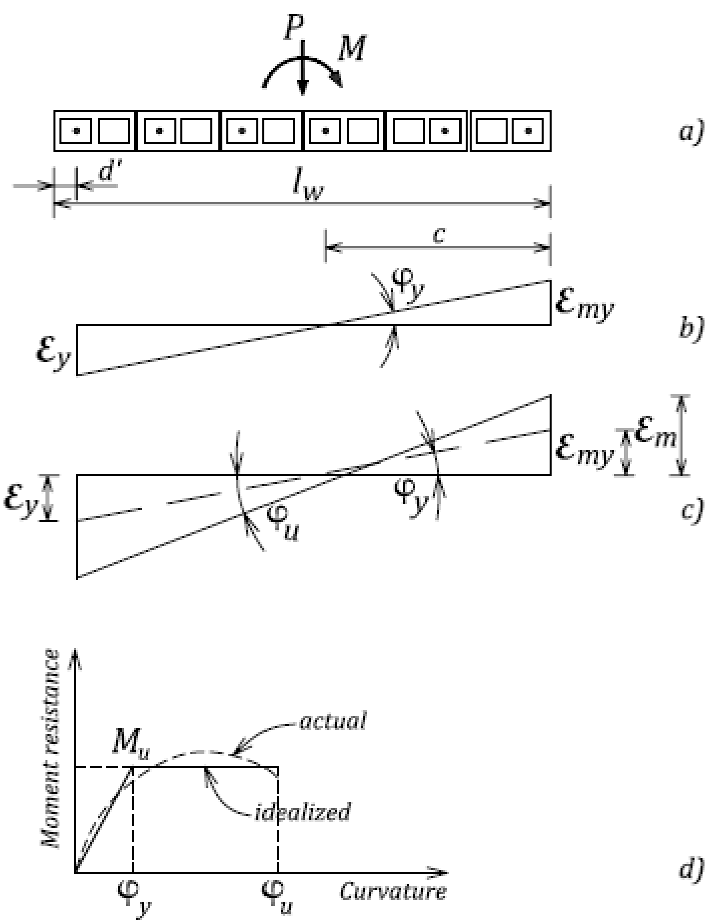


Figure B-4. Curvature in a reinforced masonry wall section: a) wall cross section; b) yield curvature; c) ultimate curvature; d) moment-curvature relationship.

The curvature value depends on the load level, the section geometry, the amount and distribution of reinforcement, and the mechanical properties of steel and masonry. An actual moment-curvature relationship for ductile sections is nonlinear, however it is usually idealized by elastic-plastic (bilinear) relationship, as shown in Figure B-4d.

Once the curvatures at the critical stages have been determined, the *curvature ductility ratio* μ_φ can be found as follows

$$\mu_\varphi = \frac{\varphi_u}{\varphi_y}$$

When the curvature distribution along a structural member (e.g. shear wall) is defined, rotations and deflections can be calculated by integrating the curvatures along the member. This can be accomplished in several ways, including the moment area method.

Rotations and deflections in a masonry shear wall at the ultimate state can be determined following the approach outlined above. Consider a cantilevered shear wall of length l_w and height h_w, and the plastic hinge length l_p (see Figure B-5a). The wall is subjected to a seismic shear force at the top, which results in a corresponding bending moment diagram as shown in Figure B-5b. The curvature diagram shown in Figure B-5c has two distinct portions: an elastic portion, with the maximum curvature equal to the yield curvature φ_y, and the plastic portion with the maximum curvature equal to the ultimate curvature φ_u. Note that the elastic portion of the curvature diagram has the same shape as the bending moment diagram (since the curvatures and bending moments are directly proportional). The actual curvature distribution in the plastic region varies in a nonlinear manner, as shown in Figure B-5c. For design purposes, the curvature can be taken as constant over the plastic hinge length l_p (note that the areas under the actual and the equivalent plastic curvature are set to be equal). The elastic rotation θ_e and the plastic rotation θ_p are presented in Figure B-5d. The plastic rotation can be determined as the area of the equivalent rectangle of width $\varphi_u - \varphi_y$ and height l_p, as shown in Figure B-5c. These rotations can be calculated from the curvature diagram as follows:

$$\theta_u = \theta_e + \theta_p$$

where

$$\theta_e = \frac{\varphi_y \cdot h_w}{2}$$

$$\theta_p = (\varphi_u - \varphi_y) \cdot l_p$$

The maximum deflection Δ_u at the top of the wall is shown in Figure B-5d. This deflection has two components: elastic deflection Δ_y corresponding to the yield curvature φ_y, and the plastic deflection Δ_p due to a rigid body rotation, since bending moments do not increase once the yielding has taken place. Deflection values can be found by taking the moment of the curvature area around point A, as follows:

$$\Delta_y = \frac{\varphi_y h_w}{2} \cdot \frac{2h_w}{3} = \frac{\varphi_y h_w{}^2}{3}$$

$$\Delta_p = (\varphi_u - \varphi_y) \cdot l_p (h_w - 0.5 l_p)$$

$$\Delta_u = \Delta_y + \Delta_p$$

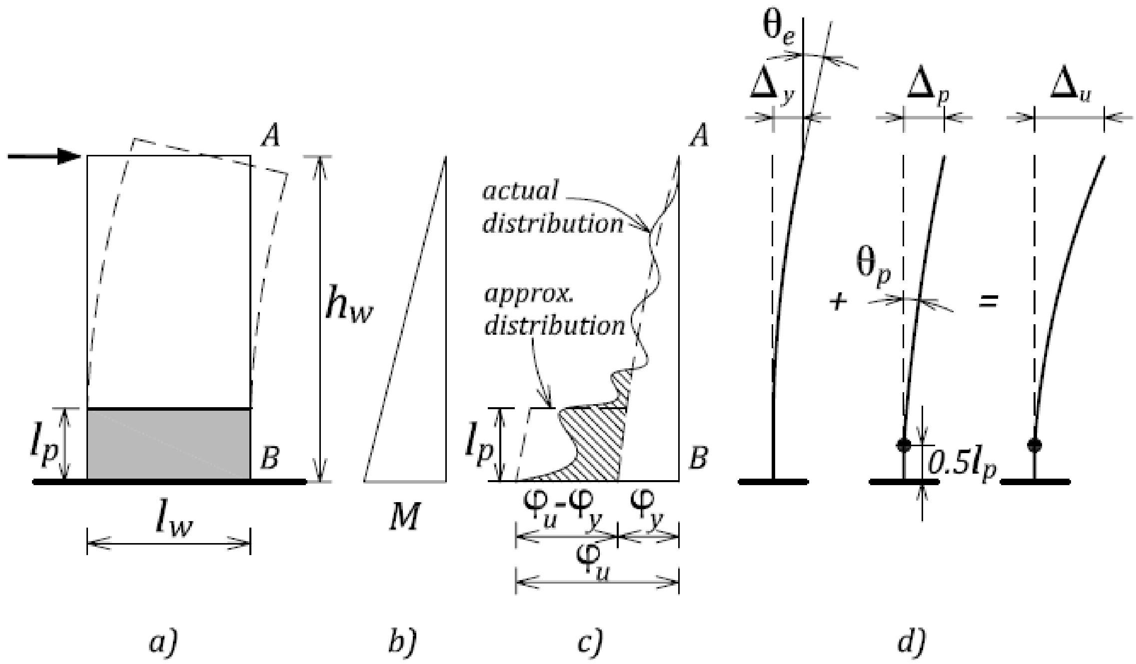


Figure B-5. Shear wall at the ultimate: a) wall elevation; b) bending moment diagram; c) curvature diagram; d) deflections.

The above equations can be used to determine the displacement ductility ratio μ_Δ, in terms of the curvature ductility μ_φ and other parameters, as follows:

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} = 1 + 3(\mu_\varphi - 1)\left(\frac{l_p}{h_w}\right)\left(1 - 0.5\frac{l_p}{h_w}\right)$$

Alternatively, the curvature ductility ratio μ_φ can be expressed in terms of the displacement ductility ratio, as follows:

$$\mu_\varphi = \frac{\varphi_u}{\varphi_y} = \frac{h_w{}^2(\mu_\Delta - 1)}{3l_p(h_w - 0.5l_p)} + 1$$

It should be noted that μ_Δ and μ_φ values are different for the same member. Once the yielding has taken place, the deformations concentrate at the plastic hinges, so the curvature ductility μ_φ is expected to be larger than the displacement ductility μ_Δ. This difference is more pronounced in walls with larger displacement ductility ratios.

B.3 Ductility Check

CSA S304.1-04 prescribes ductility check for certain classes of ductile masonry shear walls, as discussed in Section 2.5.4.3 of this document. Masonry design standards in other countries also contain ductility check provisions. For example, the New Zealand Masonry Standard NZS 4230:2004 (SANZ, 2004) Cl. 7.4.6 prescribes the c/l_w limit of 0.2 for limited ductile cantilever walls (provided that $h_w/l_w < 3$). The same limit was prescribed by the 1994 version of CSA S304.1. (Note that limited ductility walls according to the NZS 4230 are characterized by the displacement ductility of 2.0). It should be noted that the NZS 4230 prescribes maximum strain limits for unconfined and confined masonry of 0.003 and 0.008 respectively. The standard also includes a provision for confining plates in plastic hinge regions as a means of confining the compression zone of the wall section and enhancing its ductile performance (NZS 4230:2004 Cl.7.4.6.5). The seismic design provisions for reinforced concrete shear walls in CSA A23.3-04 also prescribe c/l_w limits for shear walls at different ductility levels.

B.4 Wall Height-to-Thickness Ratio Restrictions

Paulay and Priestley (1992, 1993) developed an analytical model, which offers a means to find the minimum wall thickness required to avoid out-of-plane instability. This thickness depends on several parameters, including the vertical reinforcement ratio, desired curvature and displacement ductility ratios, plastic hinge length, and the mechanical properties of steel and masonry. Paulay and Priestley also performed an experimental study to confirm their analytical model. They tested a few reinforced concrete shear wall specimens and a concrete masonry wall specimen. The masonry wall specimen failed by out-of-plane buckling at a very large displacement ductility μ_Δ of around 14.

The application of this procedure will be illustrated on an example of a reinforced masonry wall. The equation for the critical wall thickness b_c is as follows (Paulay and Priestley, 1992)

$$b_c = 0.022 l_w \sqrt{\mu_\varphi}$$

Curvature ductility, μ_φ, is related to displacement ductility, μ_Δ, as shown in Section B.3. The plastic hinge length l_p is taken equal to $h_w/6$, and so the equation can be simplified as follows

$$\mu_\varphi = 2.2(\mu_\Delta - 1)$$

The displacement ductility ratio μ_Δ can be considered equal to R_d prescribed by NBCC 2005 for different SFRSs (note that μ_Δ values in the range from 2.0 to 3.0 are considered in this example). By following the above procedure, it is possible to obtain the b_c/l_w ratios corresponding to different μ_Δ values. The results are summarized in Table B-1.

For example, if the wall length l_w is equal to 5,000 mm, the corresponding critical thickness b_c is equal to 150 mm for μ_Δ = 2.0, or 230 mm for μ_Δ = 3.0. Paulay and Priestley suggest that the critical wall thickness should be expressed as a fraction of the wall length rather than its height.

Table B-1. Critical Wall Thickness b_c Versus the Displacement Ductility Ratio μ_Δ

μ_Δ	μ_φ	l_w/b_c
2.0	2.2	31
2.5	3.3	25
3.0	4.4	22

Findings of this research were incorporated in the seismic design provisions for reinforced concrete shear walls in New Zealand and Canada (CSA A23.3 first introduced these provisions in its 1994 edition). The New Zealand masonry design standard (NZS 4230:2004) also includes provisions, which restrict the thickness of reinforced masonry shear walls; however these provisions are somewhat less stringent than the current Canadian provisions. NZS 4230:2004 prescribes the following minimum thicknesses for limited ductility walls (μ_Δ of 2.0) and ductile walls (μ_Δ of 4.0):

1. For *walls up to 3 storeys high* (Cl.7.4.4.1 and 7.3.3), minimum thickness t should not be less than $L_n/20$ (or $0.05L_n$), where L_n denotes clear vertical distance between lines of effective horizontal support or clear horizontal distance between lines of effective vertical

support. Commentary to Cl.7.3.3 states that "for a given wall thickness, t, and the case when lines of horizontal support have a clear vertical spacing of $L_n > 20t$, then vertical lines of support having a clear horizontal spacing of $L_n < 20t$ shall be provided."

2. For *walls more than 3 storeys high* (Cl.7.4.4.1) minimum thickness t shall not be less than $L_n/13.3$ (or $0.075L_n$). However, a larger wall thickness can be used provided that one of the following conditions is satisfied (maximum strain in masonry ε_u is equal to 0.003 according to NZS 4230:2004) (see Figure 2-28):
   a) $c \leq 4t$ or
   b) $c \leq 0.3l_w$ or
   c) $c \leq 6t$ from the inside of a wall return of a flanged wall, which has a minimum length $0.2L_n$.

The relaxed thickness requirement applies to the cases where the neutral axis depth is small, and so the compressed area may be so small that the adjacent vertical strips of the wall will be able to stabilize it. This is likely the case with rectangular walls subjected to low axial compression. (The same criteria for relaxed thickness restrictions are included in the seismic provisions for reinforced concrete design CSA A23.3-04 Cl.21.6.3.)

Commentary to NZS 4230 Cl.7.4.4.1 states that it is considered unlikely that failure due to lateral instability of the wall will occur in structures less than 3 storeys high, because of the rapid reduction in flexural compression with height. This is also in line with the statement made by Paulay (1986), that out-of-plane stability is likely to take place in walls with large plastic hinge length (one storey or more). According to CSA S304.1 Cl.10.16, plastic hinge length is related to the wall height (on the order of $h_w/6$), and so a large plastic hinge length would not be expected in shear walls found in low-rise masonry buildings.

Paulay and Priestley (1992) stated that "where the wall height is less than three storeys, a greater slenderness should be acceptable. In such cases, or where inelastic flexural deformations cannot develop, the wall thickness t need not be less than $0.05L_n$" (where L_n denotes clear wall length between the supports).

FEMA 306 (1999) also discusses the issue of wall instability. This document also refers to the procedure by Paulay and Priestley (1993) and provides the following recommendation for minimum wall thickness in ductile walls (μ_Δ of 4.0):

$t \leq l_w/24$ or $t \leq h/18$

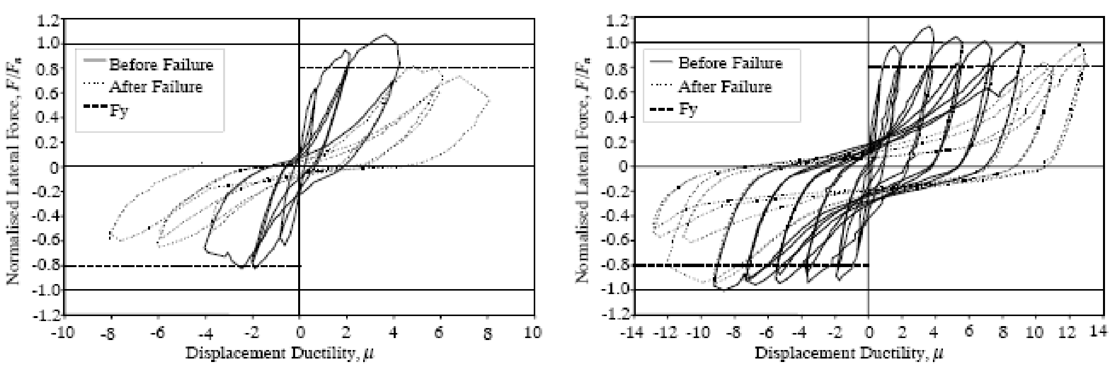
Note that the above requirement, which applies to the walls with displacement ductility ratio (μ_Δ) equal to 4.0, is the same as the CSA S304.1-04 requirement for limited ductility walls with R_d equal to 1.5.

FEMA 306 (1999) also points out that "the lack of evidence for this type of failure in existing structures may be due to the large number of cycles at high ductility that must be achieved – most conventionally designed masonry walls are likely to experience other behaviour modes such as diagonal shear before instability becomes a problem."

B.5 Grouting

Limited experimental research evidence indicates that fully grouted reinforced masonry walls demonstrate higher ductility and strength under cyclic lateral loads than otherwise similar partially grouted specimens. Ingham et al. (2001) reported the results of an experimental study of twelve full-scale reinforced masonry wall specimens subjected to an in-plane cyclic lateral load. Of the twelve specimens, nine were partially grouted, and three were fully grouted. The walls were reinforced with 12 mm diameter vertical reinforcing bars spaced at 800 mm on centre (25% grouted cores), with a bond beam at the top of the wall. The wall thickness varied from 90 mm to 190 mm, which resulted in height/length aspect ratios ranging from 0.57 to 1.33. The walls were not subjected to any external axial load. The walls were designed to fail in the diagonal shear mode. The test results showed that the fully grouted wall specimens demonstrated significantly higher displacement ductility (on the order of 6.0) than the otherwise identical partially grouted specimens (4.0). It should be noted that all of the partially grouted specimens achieved a displacement ductility of 2.0 or higher. A possible reason for the higher ductility in the fully grouted wall specimens is that they ultimately failed in the sliding shear mode, which is characterized by large deformations at the base of the wall. The partially grouted specimens failed in the shear/diagonal tension mode. Force-displacement responses for a partially grouted Wall 2 and a fully grouted Wall 3 specimen are shown in Figure B-6. Note that the specimen dimensions were identical: 2600 mm length x 2400 mm height x 100 mm nominal thickness.

It is important to note that none of the twelve specimens exhibited a sudden failure, as is typically associated with conventional (diagonal tension) shear failure; instead, gradual strength degradation was observed. The findings of related experimental studies by Voon and Ingham (2006) and Schultz (1996) were reported in Section B.1.



(a) Wall 2, 2600 x 2400 x 100, Partial Grout-filled

(b) Wall 3, 2600 x 2400 x 100, Solid Grout-filled

Figure B-6. Force-displacement responses for partially grouted (left) and fully grouted (right) wall specimens (Ingham et al., 2001, reproduced by permission of the Masonry Society).

TABLE OF CONTENTS – APPENDIX C

C Relevant Design Background

This appendix contains additional information relevant for masonry design as discussed in Chapter 2, but it is not directly related to the seismic design provisions of CSA S304.1-04. Applications of design methods and procedures presented in this appendix can be found in Chapter 4, which contains several design examples. The appendix addresses in detail a few topics of interest to masonry designers, e.g., the calculation of in-plane wall stiffness including the effect of cracking, and force distribution in perforated shear walls. However, modeling and analysis of multi-storey perforated shear walls have not been covered in this document.

C.1 Design for Combined Axial Load and Flexure

C.1.1 Reinforced Masonry Walls Under In-Plane Seismic Loading

10.2

Seismic shear forces acting at floor and roof levels cause overturning bending moments in a shear wall, which reach a maximum at the base level. In general, shear walls are subjected to the combined effects of flexure and axial gravity loads. The theory behind the design of masonry wall sections subjected to effects of flexure and axial load is well established, and is essentially the same as that of reinforced concrete walls. A typical reinforced masonry wall section is shown in Figure C-1a, along with the distribution of internal forces and strains arising from the axial load and moment. According to CSA S304.1-04, the strain distribution along the wall length is based on the assumptions that the wall section remains plane and that the maximum compressive masonry strain ε_m is equal to 0.003 (see Figure C-1b). Figure C-1c shows the distribution of internal forces on the base of the wall, as well as the axial load, P_f and the bending moment, M_f. In the compression zone, the equivalent rectangular stress block has a depth a, and a maximum stress intensity of $0.85\chi\phi_m f'_m$. Note that the χ factor assumes the value of 1.0 for members subjected to the compression perpendicular to the bed joints, such as structural walls (S304.1 Cl.10.2.6). Each reinforcing bar develops an internal force (either tension or compression), equal to the product of the factored stress and the corresponding bar area. The internal vertical forces must be in equilibrium with P_f, and the factored moment capacity M_r can be determined by taking the sum of moments of the internal forces around the centroid of the section.

The following three design scenarios and the related simplified design procedures will be discussed in this section:

1. Wall reinforcement (both concentrated and distributed) and axial load are given – find moment capacity
2. Wall is reinforced with distributed reinforcement only – find moment capacity
3. Wall reinforcement needs to be estimated (factored bending moment and axial force are given)

The first two are applicable for the common situations where a designer assumes the minimum seismic reinforcement amount and desires to find its moment capacity.

Approximate design approaches that can be used to assist designers in each of these scenarios are presented below. For detailed analysis and design procedures, the reader is referred to Drysdale and Hamid (2005) and Hatzinikolas and Korany (2005).

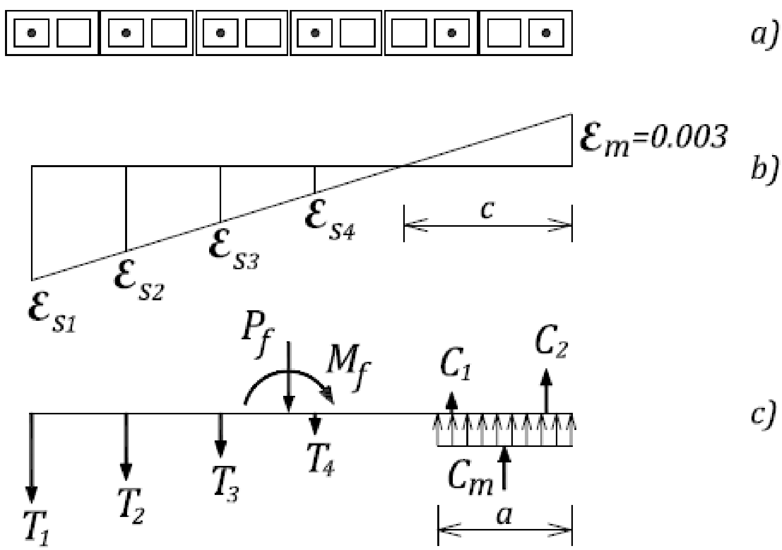


Figure C-1. A reinforced masonry shear wall under the combined effects of axial load and flexure: a) plan view cross section; b) strain distribution; c) internal force distribution.

C.1.1.1 Moment capacity for the section with concentrated and distributed reinforcement

Rectangular section

A simplified wall design model is shown in Figure C-2. The wall reinforcement can be divided into:

- Concentrated reinforcement at the ends (area A_c at each end), and
- Distributed reinforcement along the wall length (total area A_d).

It is assumed that the concentrated wall reinforcement yields either in tension or in compression at the wall ends. Also, it is assumed that the distributed reinforcement yields in tension.

A procedure to find the factored moment capacity M_r for a shear wall with a given vertical reinforcement (size and spacing) is outlined below.

From the equilibrium of vertical forces (see Figure C-2b), it follows that

$$P_f + T_1 + T_2 - C_3 - C_m = 0 \qquad (1)$$

where

$$T_1 = C_3 = \phi_s f_y A_c$$

$$T_2 = \phi_s f_y A_d$$

$$C_m = (0.85\phi_m f'_m)(t \cdot a)$$

The compression zone depth, a, can be determined from equation 1 as follows

$$a = \frac{P_f + \phi_s f_y A_d}{0.85\phi_m f'_m t} \qquad (2)$$

$\beta_1 = 0.8$ when $f'_m < 20$ MPa (note that β_1 value decreases when $f'_m > 20$ MPa, as prescribed in S304.1 Cl.10.2.6)

The neutral axis depth, c, measured from the extreme compression fibre to the point of zero strain is given by

$$c = a/\beta_1$$

Next, the factored moment capacity, M_r, can be determined by summing up the moments around the centroid of the wall section (point **O**) as follows

$$M_r = C_m(l_w - a)/2 + 2\left[\phi_s f_y A_c \left(l_w/2 - d'\right)\right] \qquad (3)$$

where d' is the distance from the extreme compression fibre to the centroid of the concentrated compression reinforcement.

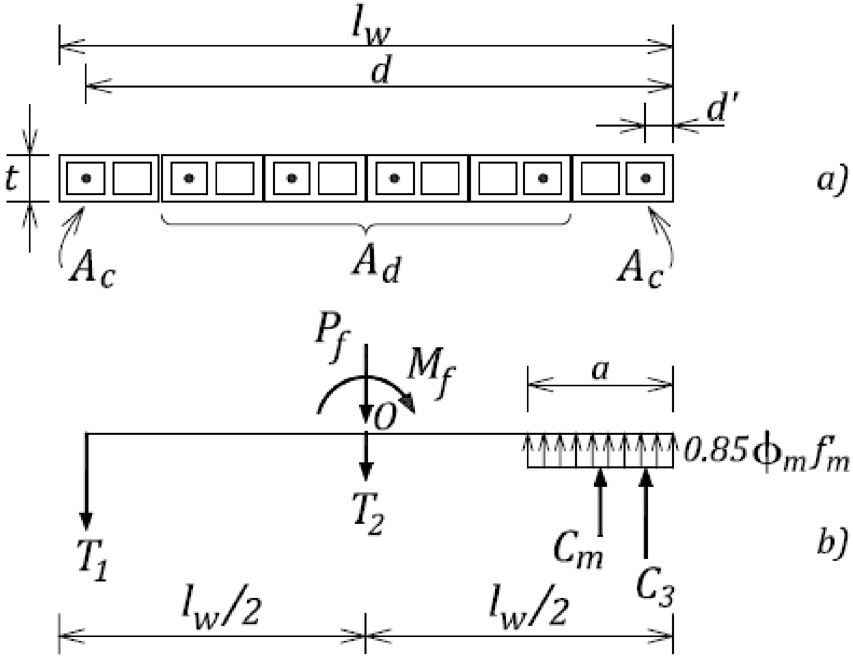


Figure C-2. A simplified design model for rectangular wall section: a) plan view cross-section showing reinforcement; b) internal force distribution.

10.2.8

In case of squat shear walls, CSA S304.1-04 prescribes the use of a reduced effective depth d for flexural design, i.e.

$$d = 0.67 l_w \leq 0.7h$$

As a result, the moment capacity should be reduced by taking a smaller lever arm for the tensile steel, as follows

$$M_r = C_m(l_w - a)/2 + \left[\phi_s f_y A_c \left(l_w/2 - d'\right)\right] + \left[\phi_s f_y A_c \left(d - l_w/2\right)\right] \qquad (4)$$

Note that the reinforcement area A_c in squat walls should be increased to provide more than one reinforcing bar, since the end zone constitutes a larger portion of the overall wall length in these cases.

The CSA S304.1-04 provision for the reduced effective depth in squat walls contained in Cl.10.2.8 is intended to account for the effect of the deep beam behaviour of squat walls. This provision makes more sense for non-seismic design, and it should not be used if the tension steel yields in seismic conditions.

Flanged section

In case of the flanged wall section shown in Figure C- 3, the factored moment capacity M_r can be determined by summing up the moments around the centroid of the wall section (point **O**) as follows

$$M_r = C_m\left(l_w/2 - x\right) + 2\left(\phi_s f_y A_c\right)\left(l_w/2 - d'\right)$$

where

$$a = \frac{A_L - b_f * t + t^2}{t}$$

$$x = \frac{t*\left(a^2/2\right) + (b_f - t)\left(t^2/2\right)}{A_L}$$

A_L is the area of the compression zone.

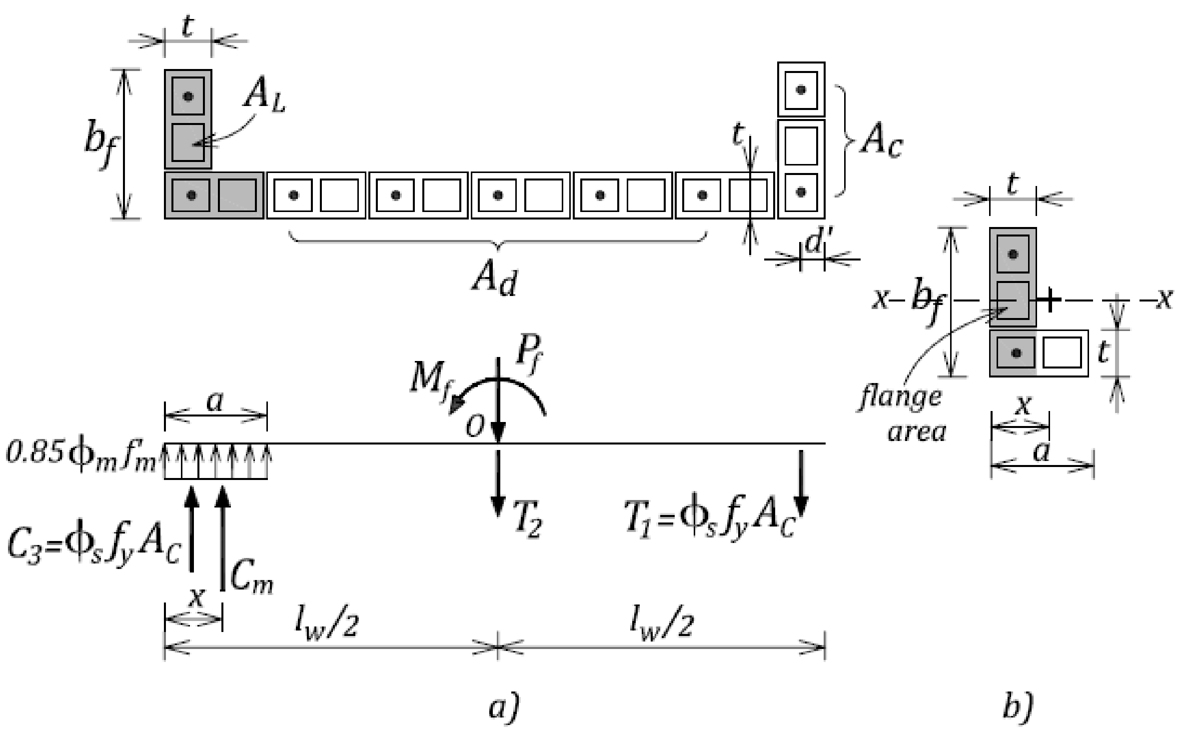


Figure C- 3. A simplified design model for flanged wall section.

C.1.1.2 Moment capacity for rectangular wall sections with distributed vertical reinforcement

The previous section discussed a general case of a shear wall with both concentrated and distributed vertical reinforcement. In low- to medium-rise concrete and masonry wall structures, the provision of distributed vertical reinforcement is often sufficient to resist the effects of combined flexure and axial loads (see Figure C-4a). The factored moment capacity for walls with distributed vertical reinforcement can be determined based on the approximate equation proposed by Cardenas and Magura (1973), which was originally developed for reinforced concrete shear walls. The equation was derived based on the assumption that the distributed wall reinforcement shown in Figure C-4b can be modeled like a thin plate of length l_w (equal to the wall length), and the thickness is such that the total area A_{vt} is the same as that provided by distributed reinforcement along the wall length (see Figure C-4b). The factored moment capacity can be determined as follows:

$$M_r = 0.5\phi_s f_y A_{vt} l_w \left(1 + \frac{P_f}{\phi_s f_y A_{vt}}\right)\left(1 - \frac{c}{l_w}\right) \qquad (5)$$

where

A_{vt} - the total area of distributed vertical reinforcement

c - neutral axis depth

$$\omega = \frac{\phi_s f_y A_{vt}}{\phi_m f'_m l_w t}$$

$$\alpha = \frac{P_f}{\phi_m f'_m l_w t}$$

$$\frac{c}{l_w} = \frac{\omega + \alpha}{2\omega + \alpha_1 \beta_1}$$

$\alpha_1 = 0.85$ and $\beta_1 = 0.8$

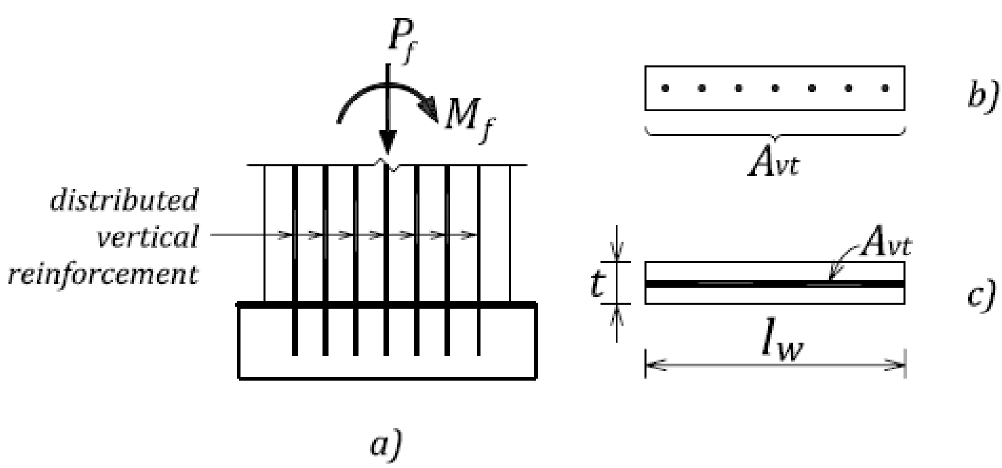


Figure C-4. Shear wall with distributed vertical reinforcement: a) vertical elevation; b) actual cross section; c) equivalent cross-section.

C.1.1.3 An approximate method to estimate the wall reinforcement

Consider a wall cross-section shown in Figure C-5a. In design practice, there is often a need to produce a quick estimate of wall reinforcement based on the given factored loads. In this case, the loads consist of the factored bending moment M_f and axial force P_f acting at the centroid of the wall section (point **O**).

The goal of this procedure is to find the total area of wall reinforcement A_s. To simplify the calculations, an assumption is made that the reinforcement yields in tension and that the resultant force T_r acts at the centroid of the wall section, that is, (see Figure C-5b)

$$T_r = \phi_s f_y A_s \qquad (6)$$

An initial estimate for the compression zone depth a can be made as follows

$$a \cong 0.3 l_w$$

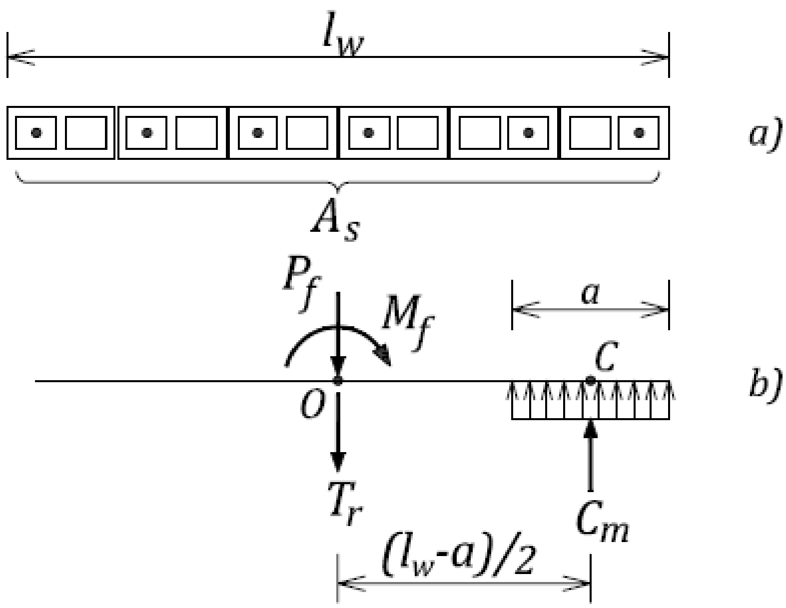


Figure C-5. Reinforcement estimate: a) plan view wall cross-section; b) distribution of internal forces.

Next, compute the sum of moments of all forces around the centroid of the compression zone (point **C**), as follows

$$M_f - P_f (l_w - a)/2 - T_r (l_w - a)/2 = 0$$

From the above equation it follows that

$$T_r = \frac{M_f - P_f (l_w - a)/2}{(l_w - a)/2} \qquad (7)$$

The area of reinforcement can then be determined from equation (7) as follows

$$A_s = T_r / \phi_s f_y$$

The area of reinforcement can be chosen to be equal to or larger than that estimated by this procedure. A uniform reinforcement distribution over the wall length is recommended for seismic design, since research studies have shown that shear walls with uniform reinforcement distribution show better seismic response in the post-cracking range. In addition, the seismic detailing requirements for vertical reinforcement need to be followed.

C.1.2 Reinforced Masonry Walls Under Out-of-Plane Seismic Loading

Masonry walls are subjected to the effects of seismic loads acting perpendicular to their surface – this is called *out-of-plane seismic loading*. For design purposes, wall strips of a predefined width are treated as beams spanning vertically or horizontally between lateral supports. When the walls span in the vertical direction, floor and/or roof diaphragms provide the lateral supports.

Walls can also span horizontally, in which case the lateral supports need to be provided by cross walls or pilasters, as shown in Figure C-6. Note that support on four edges is very efficient, since these walls behave as two-way slabs.

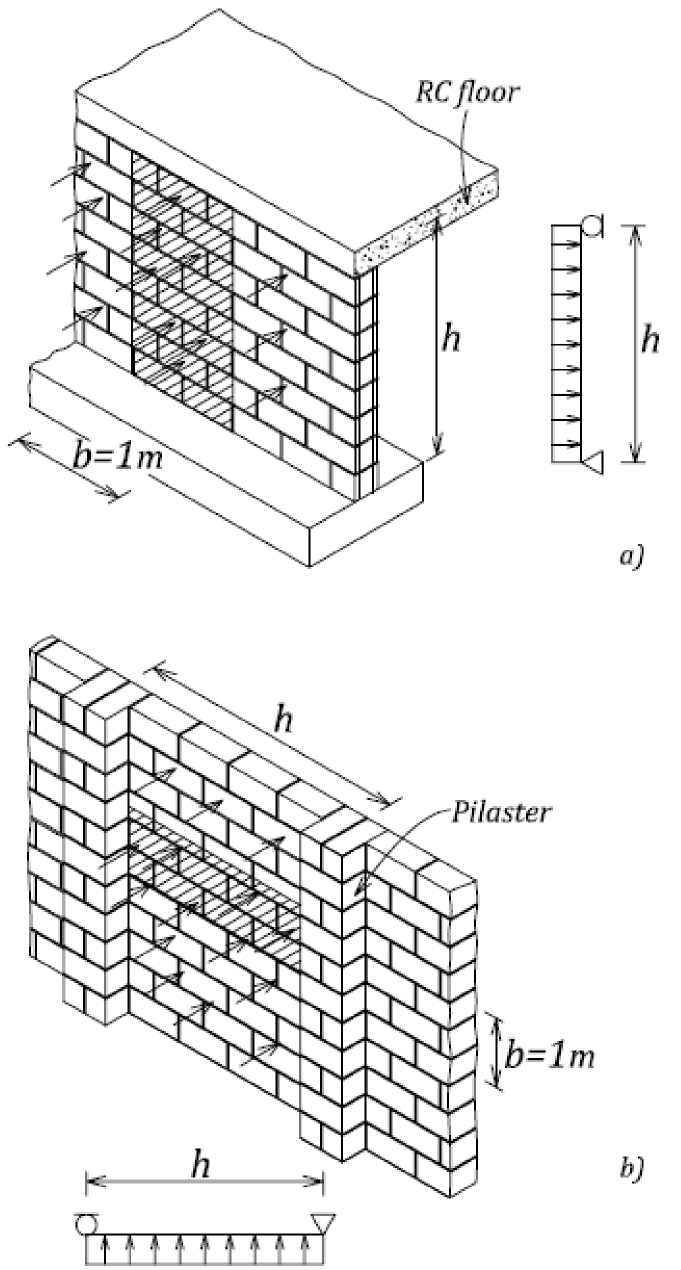


Figure C-6. Masonry walls under out-of plane seismic loads: a) spanning vertically between floor/roof diaphragms; b) spanning horizontally between pilasters.

Consider a reinforced concrete masonry wall subjected to the effects of factored axial load P_f and bending moment M_f, as shown in Figure C-7a. The wall is reinforced vertically, with only the reinforced cores grouted. It is assumed that the size and distribution of vertical reinforcement are given. The notation used in Figure C-7b is explained below:

t - overall wall thickness (taken as actual block width, e.g. 140 mm, 190 mm, etc.)

t_f - face shell thickness

b - effective width of the compression zone (see Section 2.4.2 and Figure 2-19)

d - effective depth, that is, distance from the extreme compression fibre to the centroid of the wall reinforcement; typically, the reinforcement is placed in the middle of the wall section, so

$d = t/2$

A_s - total area of steel reinforcement placed within the effective width b

It is assumed that the steel has yielded, that is, $\varepsilon_s \geq \varepsilon_y$, and the corresponding stress in the reinforcement is equal to the yield stress, f_y. This is a reasonable assumption for low-rise masonry buildings, since the axial load is low and the walls are expected to fail in the steel-controlled mode. The design procedure is outlined below.

- The resultant forces in steel T_r and masonry C_m can be determined as follows:

  $T_r = \phi_s f_y A_s$

  $C_m = (0.85\phi_m f'_m)(b \cdot a)$
- The equation of equilibrium of internal forces gives (see Figure C-7d)

  $C_m = P_f + T_r$
- The depth of the compression stress block a is equal to

$$a = \frac{C_m}{0.85\phi_m f'_m b} \qquad (8)$$

- The moment resistance can be found from the following equation

$$M'_r = C_m (d - a/2) \qquad (9)$$

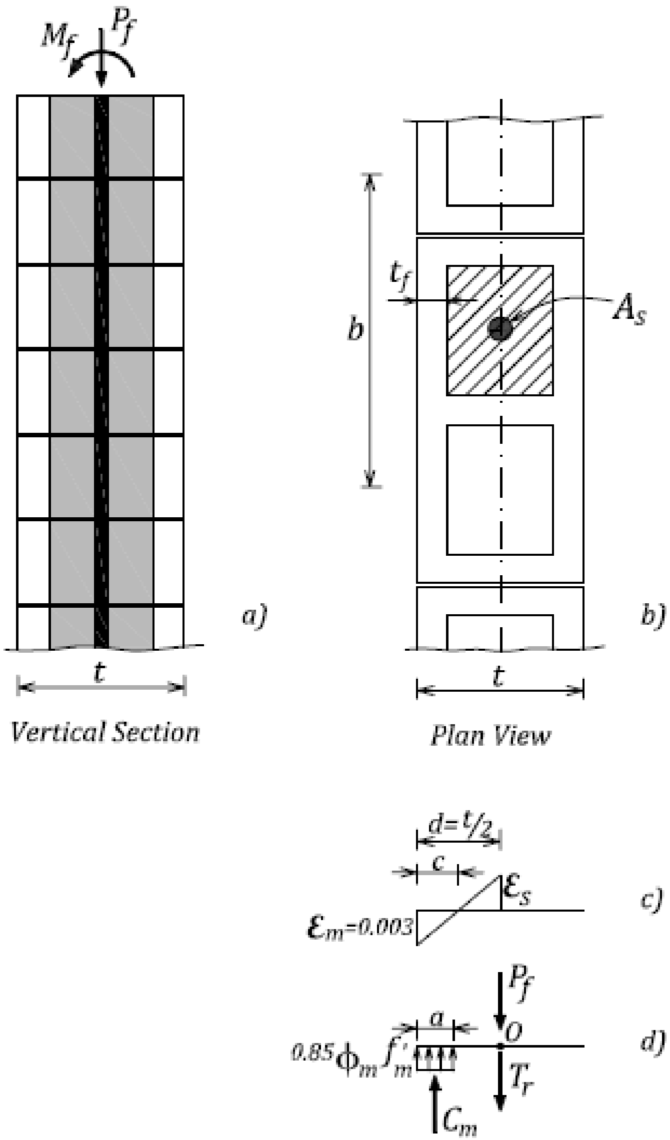


Figure C-7. A wall under axial load and out-of-plane bending: a) vertical section showing factored loads; b) plan view of a wall cross-section; c) strain distribution; d) internal force distribution.

For partially grouted wall sections (where only reinforced cores are grouted), the designer needs to confirm that

$$a \le t_f$$

When the above relation is correct, then the compression zone is rectangular, as shown in Figure C-8a. Note: in solidly grouted walls, the compression zone is always rectangular!

When $a \ge t_f$, the compression zone needs to be treated as a T-section and an additional calculation is required to determine the a value. The following equations can be used to determine the moment resistance in sections with a T-shaped compression zone:

- The resultant force in the steel T_r can be determined as follows:

$$T_r = \phi_s f_y A_s$$

- The resultant force in the masonry, C_m, acts at the centroid of the compression zone and can be determined from the equation of equilibrium of internal forces, that is,

$$C_m = P_f + T_r$$

Once the compression force in the masonry is found, the area of the masonry compression zone, A_m (see Figure C-8b), is given by

$$C_m = (0.85\phi_m f'_m) \cdot A_m$$

- The depth of the compression stress block a can be found from the following equation

$$A_m = b \cdot t_f + (a - t_f) \cdot b_w$$

where

b_w = width of the grouted cell plus the adjacent webs

- The distance from the extreme compression fibre to the centroid of the compression zone $\bar{a}$ is equal to

$$\bar{a} = \frac{b \cdot (t_f^2/2) + (a - t_f) \cdot \left(t_f + \frac{a - t_f}{2}\right)}{A_m} \qquad (10)$$

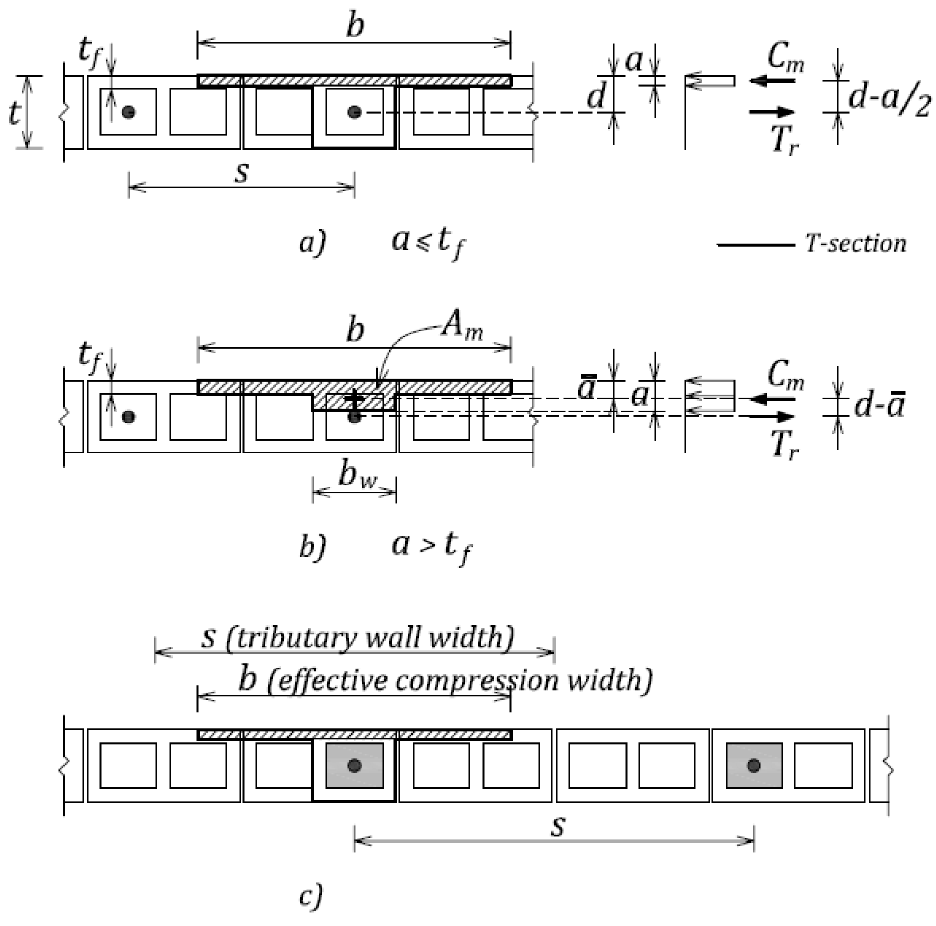


Figure C-8. Masonry compression zone: a) rectangular shape; b) T-shape; c) effective width and tributary width.

- The moment resistance can be found from the following equation

$$M'_r = C_m\left(d - \bar{a}\right) \quad (11)$$

Note that M'_r denotes the moment capacity for a wall section of width b. It is usually more practical to convert the M'_r value to a unit width equal to 1 metre (see Figure C-8c), as follows

$$M_r = M'_r\left(1.0/s\right) \quad (12)$$

where
s - spacing of vertical reinforcement expressed in metres (where $b \le s$)
M_r - factored moment capacity in kNm/m.

The design of masonry walls subjected to the combined effects of axial load and bending is often performed using P-M interaction diagrams. The axial load capacity is shown on the vertical axis of the diagram, while the moment capacity is shown on the horizontal axis. The points on the diagram represent the combinations of axial forces and bending moments corresponding to the capacity of a wall cross-section. An interaction diagram is defined by the following four distinct points and/or regions: i) balanced point, ii) points controlled by steel yielding, iii) points controlled by masonry compression, and iv) pure compression (zero eccentricity). A conceptual wall interaction diagram is presented in Figure C-9.

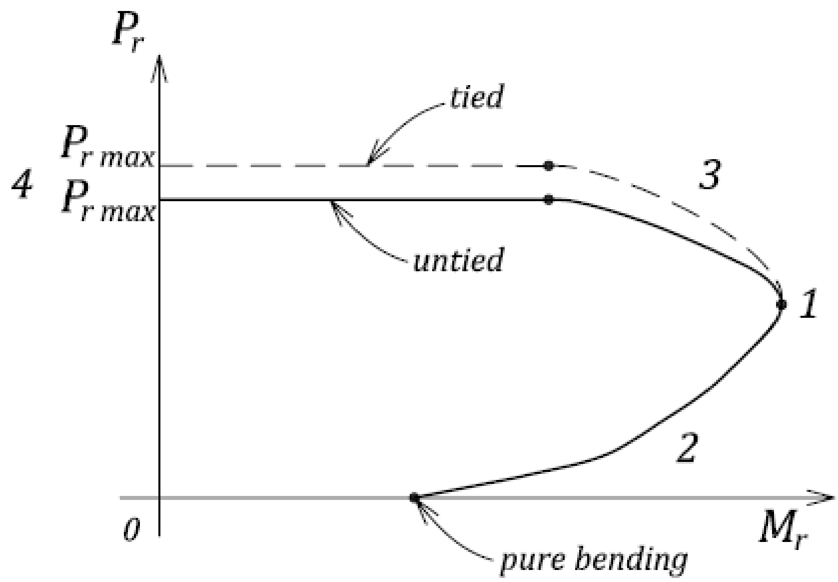


Figure C-9. P-M interaction diagram.

1. Balanced point
At the load corresponding to the balanced point, the steel has just yielded, that is, $\varepsilon_s = \varepsilon_y$. The position of the neutral axis c_b can be determined from the following proportion (see Figure C-7 c):

$$\frac{c_b}{d - c_b} = \frac{\varepsilon_m}{\varepsilon_y}$$

or

$$c_b = d\left(\frac{\varepsilon_m}{\varepsilon_m + \varepsilon_y}\right)$$

For $f_y = 400$ MPa and $\varepsilon_y = 0.002$ it follows that

$c_b = 0.6d$

2. Points controlled by steel yielding

For $c < c_b$, the steel will yield before the masonry reaches its maximum useful strain (0.003). Since the steel is yielding, it follows that $\varepsilon_s > \varepsilon_y$. The designer needs to assume the neutral axis depth (c) value so that $c < c_b$. The compression zone depth can then be calculated as $a = \beta_1 c = 0.8c$ (this is valid for $f'_m < 20MPa$ according to S304.1 Cl.10.2.6). Combinations of axial force and moment values corresponding to an assumed neutral axis depth can be found from the following equations of equilibrium (see Figure C-7d)

$P_r = C_m - T_r$

where

$T_r = \phi_s f_y A_s$ (note that the stress in the steel is equal to f_y since the steel is yielding)

Moment resistance depends on the shape of the masonry compression zone, that is, on whether the section is partially or solidly grouted.

- For a solidly grouted section or a partially grouted section with the compression zone in the face shells only:

$M'_r = C_m(d - a/2)$

where

$C_m = (0.85\phi_m f'_m)(b \cdot a)$

- For a partially grouted section with the compression zone extending into the grouted cells:

$M'_r = C_m(d - \bar{a})$

where

$C_m = (0.85\phi_m f'_m) \cdot A_m$

3. Points controlled by masonry compression

For $c > c_b$, the steel will remain elastic, that is, $\varepsilon_s < \varepsilon_y$ and $f_s < f_y$, while the masonry reaches its maximum strain of 0.003. The designer needs to assume the neutral axis depth (c) value so that $c > c_b$, and the strain in steel can then be determined from the following proportion (see Figure C-7 c):

$$\frac{\varepsilon_m}{d} = \frac{\varepsilon_s}{d - c}$$

thus

$$\varepsilon_s = \varepsilon_m \left(\frac{d - c}{c} \right)$$

The stress in the steel can be determined from Hooke's Law as follows

$f_s = E_s * \varepsilon_s$ (note that steel stress $f_s < f_y$)

where E_s is the modulus of elasticity for steel. The equations of equilibrium are the same as used in part 2 above, except that

$T_r = \phi_s f_s A_s$

The point corresponding to $c = t/2$ is considered as a special case. At that point, the strain distribution is defined by the following values

$\varepsilon_m = 0.003$ and $\varepsilon_s = 0$

thus
$T_r = 0$

4. Pure compression (zero eccentricity)
In the case of pure axial compression (S304.1 Cl.10.4) the axial load resistance for untied sections can be determined as follows:

$P_r = 0.85\phi_m f'_m A_e$ actual axial compression resistance

and

$P_{r\max} = 0.8P_r$ design axial compression resistance

According to S304.1 Cl.10.2.7, when the steel bars are tied by means of joint reinforcement, then the steel contribution can be considered for the compression resistance. The design equation for tied wall sections is as follows:

$$P_r = 0.85\phi_m f'_m (A_e - A_s) + \phi_s f_y A_s$$

and

$$P_{r\max} = 0.8P_r$$

C.2 Wall Intersections and Flanged Shear Walls

Flanged shear wall configurations are encountered when a main shear wall intersects a cross-wall (or transverse wall). Examples of flanged walls in masonry buildings are very common, since the bearing wall systems often consist of walls laid in two orthogonal directions. Also, in medium-rise wood frame apartment buildings, elevator shafts are usually of masonry construction, and the intersecting masonry walls that form the core can be considered as flanged walls.

10.6.2

In flanged shear walls, a portion of the cross wall is considered to act as the flange, while the main shear wall acts at the web. Depending on the cross-wall configuration, flanged shear walls may be of I, T- or L-section. An I-section is characterized by the two end flanges, similar to that in Figure C-10 (left), a T-section is characterized with one flanged end and other rectangular/ non-flanged end, while a L-section is characterized by one flanged end (similar to that shown in Figure C-10 right), and other rectangular-shaped (non-flanged) end. Design codes prescribe the maximum effective flange width that may be considered in the shear wall design. The CSA S304.1 requirements for overhanging flange widths for these wall sections are summarized in Table C-1 and Figure C-10. For masonry buildings with substantial flanges the height ratio limits will usually govern.

Table C-1. Overhanging Flange Width Restrictions for T- and L- Section Walls per CSA S304.1 Cl.10.6.2

T-sections (b_T)	**L-sections (b_L)**
$b_T \leq$ the smallest of:	$b_L \leq$ the smallest of:
a) b_{actual}	a) b_{actual}
b) $a_w/2$	b) $a_w/2$
c) $6 \cdot t$	c) $6 \cdot t$
d) $h_w/12$	d) $h_w/16$

where
b_{actual} - actual overhang/flange width
a_w - clear distance between the adjacent cross walls
t - actual flange thickness
h_w - wall height

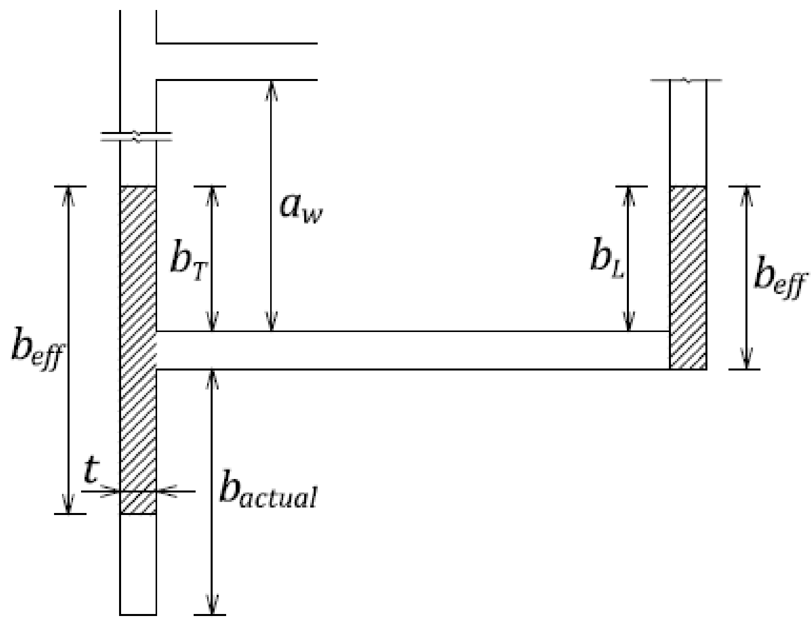


Figure C-10. CSA S304.1 flange width requirements.

7.11 10.11

Flanges do not contribute significantly to the shear resistance of flanged walls, but they generally enhance the in-plane flexural capacity. However, flanges can be considered to be effective in resisting the applied loads only if the web-to-flange joint is capable of transferring the vertical shear. According to CSA S304.1 Cl.7.11, the following alternative approaches can be used to ensure the effective shear transfer across the web-to-flange connection in both unreinforced and reinforced masonry walls (see Figure C-11):

a) Bonded intersections - 50% of the units of one wall embedded at least 90 mm in the other wall (Cl.7.11.1).
b) Mechanical connection with steel connectors (e.g. anchors, rods, or bolts) at a maximum spacing of 600 mm (Cl.7.11.3), and
c) Fully grouted keyways or recesses, with a minimum of two 3.65 mm diameter steel wires from joint reinforcement spaced at 400 mm vertically (Cl.7.11.2).

d) Fully grouted bond beam intersections with15M reinforcing bars spaced as required; this is not explicitly prescribed by CSA S304.1-04, but it is in line with the approach c) outlined in Cl.7.11.3. The bars should be detailed to develop the full yield stress on each side of the intersection.

Note that Cl.10.11.2 does not permit the use of rigid anchors (approach b) for portions of reinforced masonry shear walls in which the flanges contain tensile steel and are subject to axial tension, but alternative solutions are permitted.

Vertical shear resistance of the flanged walls must be checked by one of the following methods:

- For bonded intersections achieved by approach a), vertical shear at the intersection shall not exceed the out-of-plane masonry shear resistance (Cl.7.10.2).
- For flanged sections with the mechanical steel connectors (approach b), the connectors must be capable of resisting the vertical shear at the intersection. The connector resistance should be determined according to CSA A370-04.
- For flanged sections with the horizontal reinforcement (approaches c and d), the reinforcement must be capable of resisting the vertical shear at the intersection.

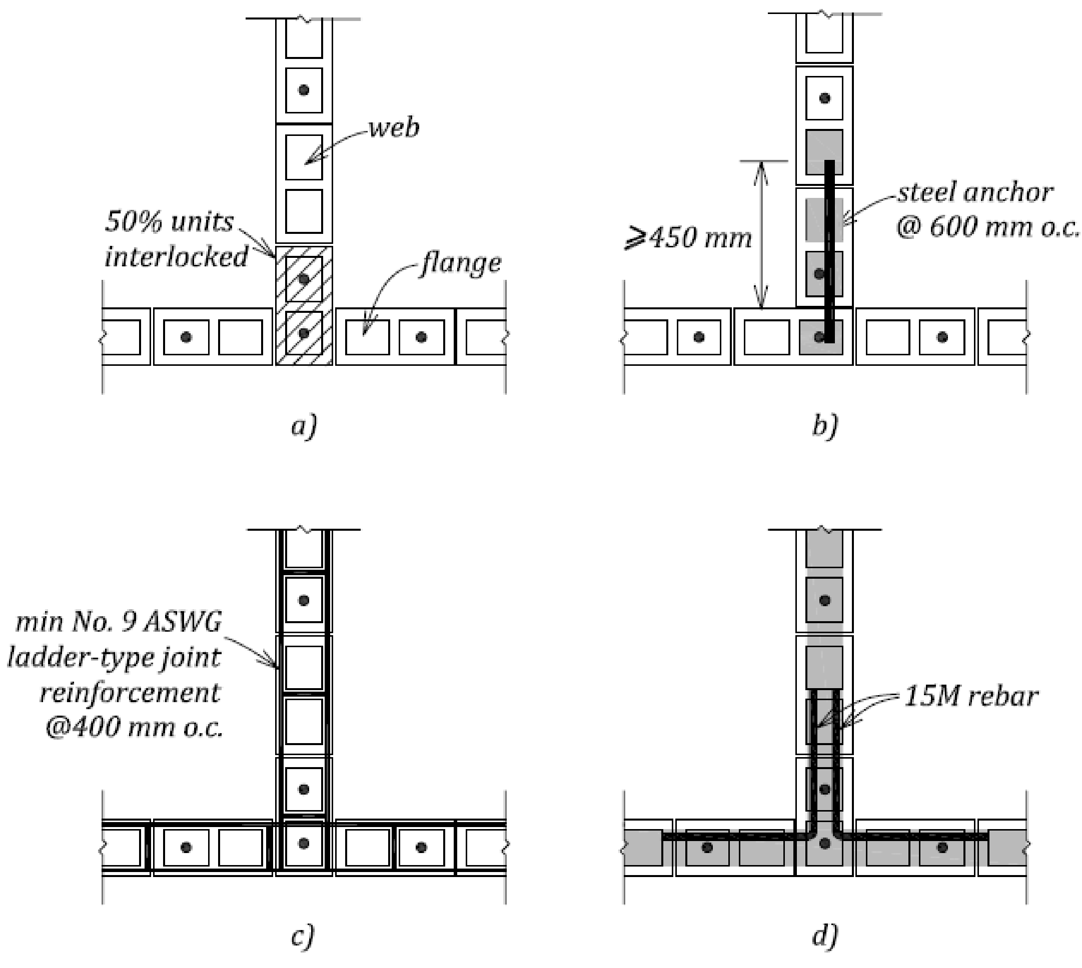


Figure C-11. Masonry wall intersections: a) bonded intersections; b) mechanical connection; c) horizontal joint reinforcement; d) horizontal reinforcing bars (bond beam reinforcement).

7.11.4

Where wall intersections are not bonded and rigid steel connectors are not used, the factored shear resistance of the web-to-flange joint shall be based on the shear friction resistance taken as

$$V_r = \phi_m \mu C_h$$

where
μ = 1.0 coefficient of friction for the web-to-flange joint
C_h = compressive force in the masonry acting normal to the head joint, normally taken as the factored tensile force at yield of the horizontal reinforcement that crosses the vertical section. The reinforcement must be detailed to enable it to develop its yield strength on both sides of the vertical masonry joint, which may be hard to achieve in practice.

Commentary

The provisions related to flanged shear walls have not changed in CSA S304.1-04 from the 1994 edition, with the exception of the new Cl.7.11.4 related to the shear friction resistance of wall intersections.

For flanged walls with horizontal reinforcement, resistance to vertical shear sliding is provided by the frictional forces between the sliding surfaces, that is, the web and the flange of the wall. The shear friction resistance V_r is proportional to the coefficient of friction μ, and the clamping force C_h acting perpendicular to the joint of height h (see Figure C-12a).

C_h is equal to the sum of the tensile yield forces developed in reinforcement of area A_b spaced at the distance s, that is,

$$C_h = \phi_s f_y A_b \, h/s$$

In case of a flanged shear wall with openings, shear friction resistance V_r is provided by wall segments between the openings, as shown in Figure C-12b.

Reinforcement providing the shear friction resistance should be distributed uniformly across the joint. The bars should be long enough so that their yield strength can be developed on both sides of the vertical joint, as shown in Figure C-13b.

Clauses 7.11.1 to 7.11.3 list three approaches (a, b, and c) that can be used to ensure shear transfer at the web-to-flange interface. In addition to the three approaches stated in CSA S304.1-04, it is a common practice in Canada to use 15M reinforcing bars from intersecting bond beams to provide shear resistance if needed (approach d). U.S. masonry design standard ACI 530-08 Cl.1.9.4.2.5 c) prescribes intersecting bond beams in intersecting walls at maximum spacing of 1200 mm (4 ft) on centre. The bond beam reinforcement area shall not be less than 200 mm^2 per metre of wall height (0.1 in^2/ft), and the reinforcement shall be detailed to develop the full yield stress at the intersection.

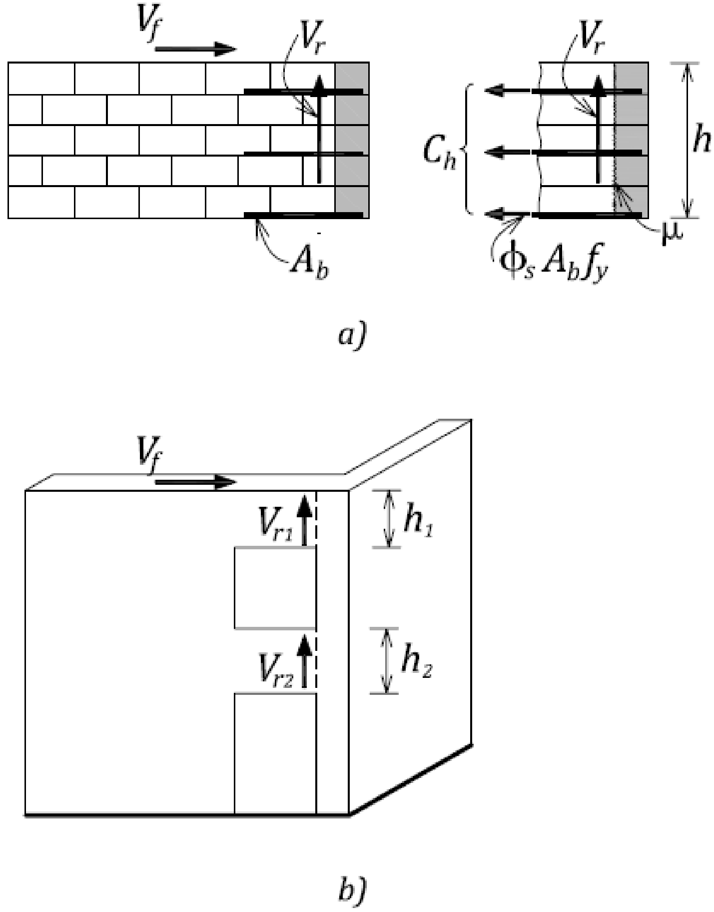


Figure C-12. Shear friction resistance at the web-to-flange intersection: a) resistance provided by the reinforcement; b) flanged shear wall with openings.

When the shear resistance of the web-to-flange interface relies on masonry only (see Figure C-13a), the horizontal shear stress v_f, due to shear force V_f, can be given by:

$$v_f = \frac{V_f}{t_e l_w}$$

where

t_e - effective web width

l_w - wall length

The designer should also find the vertical shear stress caused by the resultant compression force P_{fb}:

$$v_f = \frac{P_{fb}}{b_w * h_w}$$

The larger of these two values governs. The factored shear stress should be less than the factored masonry shear resistance, $\phi_m v_m$, as follows

$$v_f \le \phi_m v_m$$

where

$v_m = 0.16\sqrt{f'_m}$

If the above condition is not satisfied, horizontal reinforcement needs to be provided (see Figure C-13b), and the following shear resistance check should be used

$v_f \leq \phi_m v_m + v_s$

where v_s is the factored shear resistance provided by the steel reinforcement, which can be determined as follows:

$$v_s = \frac{\phi_s A_b f_y}{s \cdot t_e}$$

where A_b is area of horizontal steel reinforcement crossing the web-to-flange intersection at the spacing s.

Note that the reinforcement that crosses the vertical section has to be detailed to develop yield strength on both sides of the vertical masonry joint (see Figure C-13b).

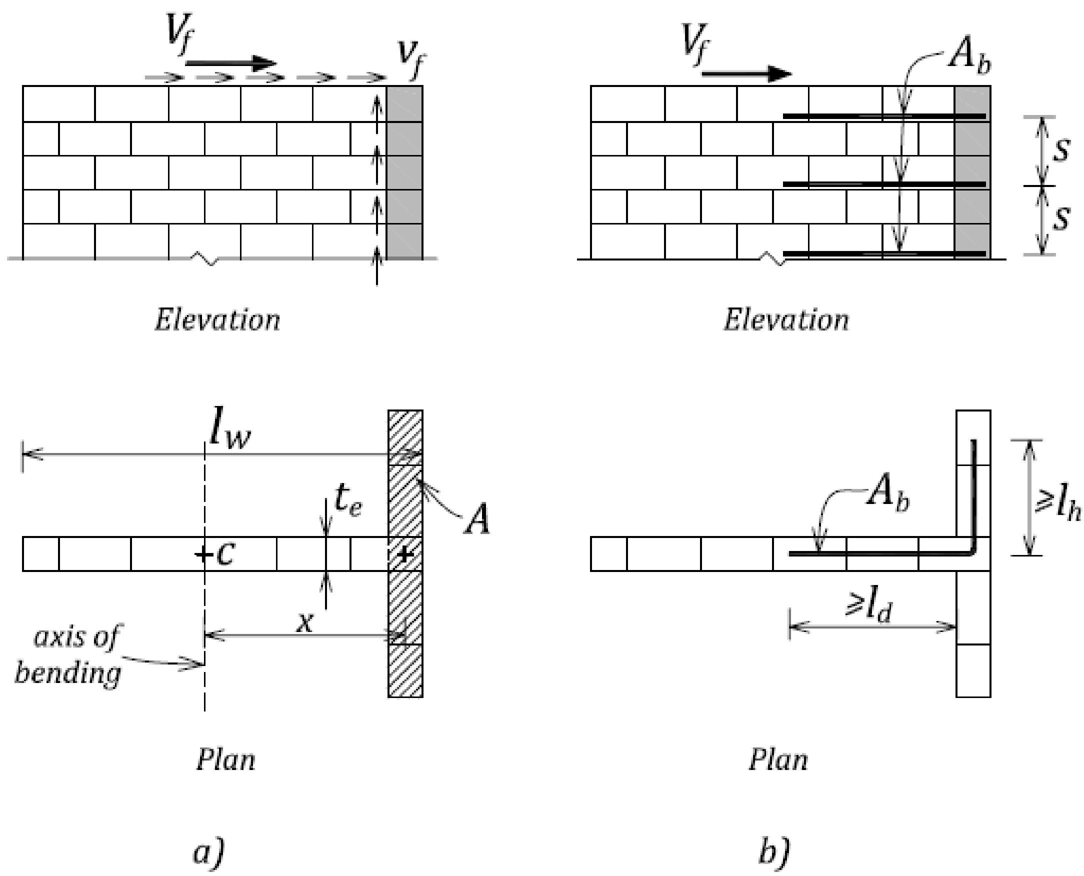


Figure C-13. Shear resistance of the web-to-flange interface: a) bonded masonry intersection; b) horizontal reinforcement at the intersection.

C.3 Wall Stiffness Calculations

The determination of wall stiffness is one of the key topics in the seismic design of masonry walls. Although this topic has been covered in other references (e.g. Drysdale and Hamid, 2006, and Hatzinikolas and Korany, 2006), a few key concepts are discussed in this section. Section C.3.2 derives expressions for the in-plane lateral stiffness of walls under the assumption that the walls are uncracked. For seismic analysis it is expected that the walls will be pushed into the nonlinear range, and so cracking will occur and the reinforcement will yield. The stiffness to be used in seismic analysis should not be the linear elastic (uncracked) stiffness but some effective stiffness that reflects the effect of cracking up to the yield capacity of the wall. Section C.3.5 gives some suggestions for the effective stiffness of shear walls responding in shear-dominant and flexure-dominant modes.

C.3.1 Lateral Load Distribution

The distribution of lateral seismic loads to individual walls can be performed once the storey shear forces have been determined from the seismic analysis. The flexibility of floor and/or roof diaphragms is one of the key factors influencing the load distribution (for more details, see Section 1.5.9 and Example 3 in Chapter 4). In the case of a flexible diaphragm, the lateral storey forces are usually distributed to the individual walls based on the tributary area. In the case of a rigid diaphragm, these forces are distributed in proportion to the stiffness of each wall. In calculating the wall forces, torsional effects must be considered, as discussed in Section 1.5.9. The distribution of lateral loads (without torsional effects) in a single-storey building with a rigid diaphragm is shown in Figure C-14.

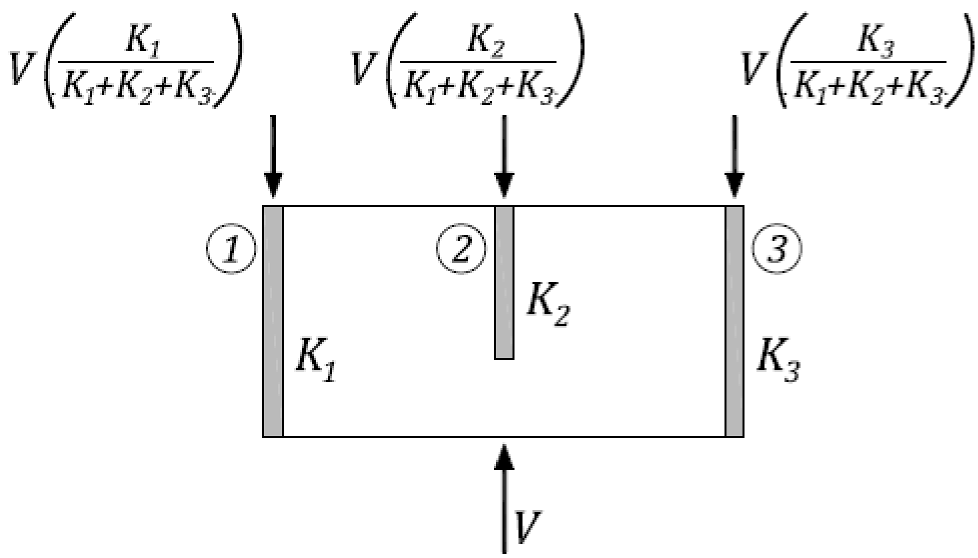


Figure C-14. Distribution of lateral loads to individual walls.

Wall stiffness is usually determined from the elastic analysis, and depends on wall height/length aspect ratio, thickness, mechanical properties, extent of cracking, size and location of openings, etc.

C.3.2 Wall Stiffness: Cantilever and Fixed-End Model

Wall stiffness depends on the end support conditions, that is, whether a wall or pier is fixed or free to move and/or rotate at its ends. Two models for wall stiffness include the cantilever model and the fixed-end model shown in Figure C-15. In the cantilever model, the wall is free to rotate and move at the top in the horizontal direction – this is usually an appropriate model for the walls in a single-storey masonry building.

The stiffness can be defined as the lateral force required to produce a unit displacement, but it is determined by taking the inverse of the combined flexural and shear displacements produced by a unit load. It should be noted that flexural displacements will govern for walls with an aspect ratio of 2 or higher. For example, the contribution of shear deformation in a wall with a height/length aspect ratio of 2.0, is 16% for the cantilever model and 43% for the fixed-end model. The stiffness equations presented in this section take into account both shear and flexural deformations.

The stiffness of a cantilever wall or a pier can be determined from the following equation (see Figure C-15 a):

$$\frac{K}{E_m * t} = \frac{1}{\left(\frac{h}{l_w}\right)\left[4\left(\frac{h}{l_w}\right)^2 + 3\right]} \qquad (13)$$

The stiffness of a wall or a pier with the fixed ends can be determined from the following equation (see Figure C-15 b):

$$\frac{K}{E_m * t} = \frac{1}{\left(\frac{h}{l_w}\right)\left[\left(\frac{h}{l_w}\right)^2 + 3\right]} \qquad (14)$$

where

h - wall height (cantilever model) or clear pier height (fixed-end model)

l_w - wall or pier length

$E_m = 850 f'_m$ modulus of elasticity for masonry

The following assumptions have been taken in deriving the above equations:

$G_m = 0.4E_m$ modulus of rigidity for masonry (shear modulus)

$I = \frac{t_e * l_w^3}{12}$ uncracked wall moment of inertia

$A_v = \frac{5 * t_e * l_w}{6}$ shear area (applies to rectangular wall sections only)

where t_e = effective wall thickness.

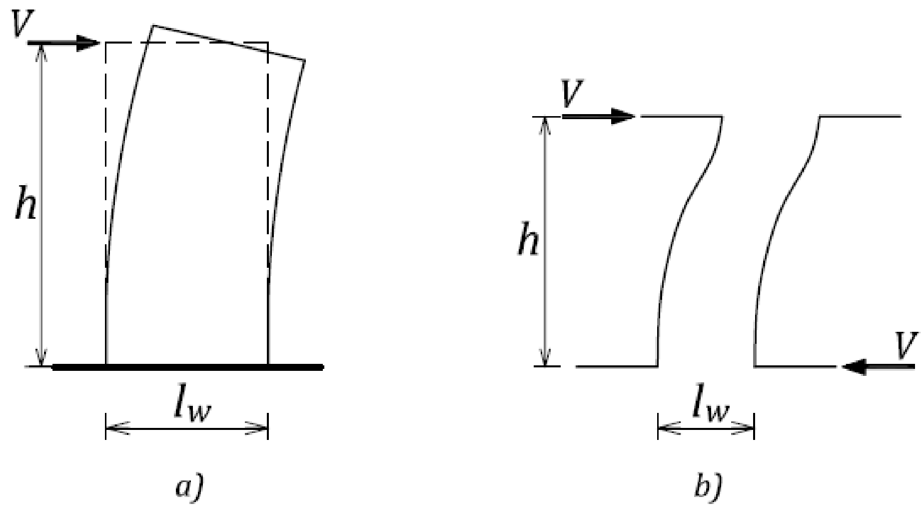


Figure C-15. Wall stiffness models: a) cantilever model, and b) fixed-end model.

The wall stiffnesses for both models for a range of height/length aspect ratios are presented in Table D-3. Note that the derivation of stiffness equations has been omitted since it can be found in other references (see Hatzinikolas and Korany, 2005).

C.3.3 Approximate Method for Force Distribution in Masonry Shear Walls

In most real-life design applications, walls are perforated with openings (doors and windows). The seismic shear force in a perforated wall can be distributed to the piers in proportion to their stiffnesses. This approach is feasible when the openings are very large and the stiffness of lintel beams is small relative to the pier stiffnesses, or if the lintel beam is very stiff so that connected piers act as fixed-ended walls. Figure C-16 illustrates the distribution of wall shear force V to individual piers in direct proportion to their stiffness. Note that, according to this model, the wall shear force is equal to the sum of shear forces in the piers, that is,

$$V = \sum V_i$$

where

$V_i = K_i * \Delta_i$ force in the pier i

Thus

$$V = \sum (K_i * \Delta_i)$$

If the floor diaphragm is considered to be rigid, it can be assumed that the lateral displacement in all piers is equal to Δ, that is,

$$\Delta_A = \Delta_B = \Delta_C = \Delta$$

and so

$$V = (\sum K_i) * \Delta$$

Thus

$$\Delta = \frac{V}{\sum K_i}$$

where

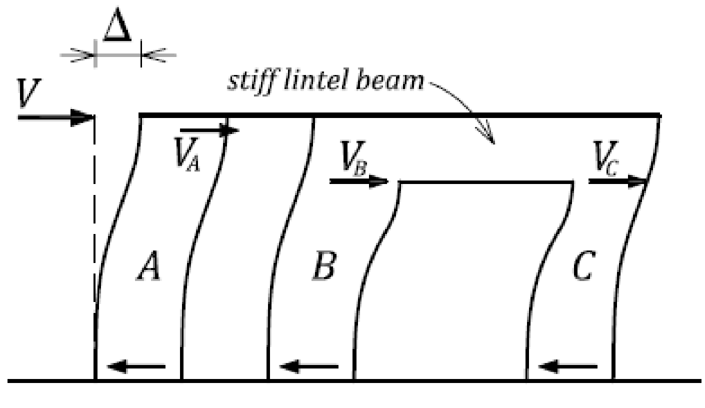
$$K = \sum K_i$$

denotes the overall wall stiffness for the system.

Therefore, the force in each pier is proportional to its stiffness relative to the sum of all pier stiffnesses within the wall, as follows

$$V_i = K_i * \Delta_i = K_i * \frac{V}{\sum K_i} = V * \frac{K_i}{\sum K_i}$$

This means that stiffer piers are going to attract a larger portion of the overall shear force. This can be explained by the fact that a larger fraction of the total lateral force is required to produce the same deflection in a stiffer wall as in a more flexible one.



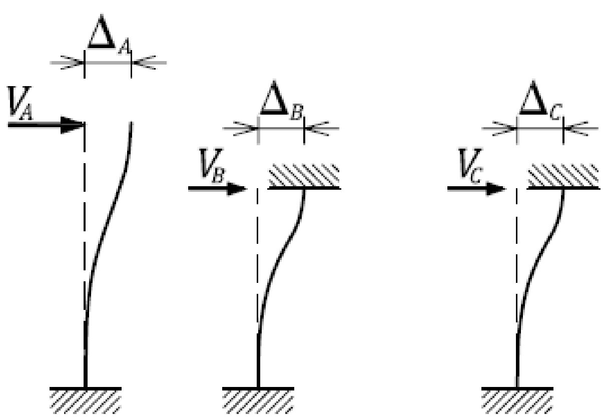


Figure C-16. Shear force distribution in a wall with a rigid diaphragm: a) wall in the deformed shape: b) pier forces.

An approximate approach for determining the stiffness of a solid shear wall in a multi-storey building is to consider the structure as an equivalent single-storey structure, as shown in Figure C-17. The entire shear force is applied at the effective height, h_e, defined as the height at which the shear force V_f must be applied to produce the base moment M_f, that is,

$$h_e = \frac{M_f}{V_f}$$

The wall stiffness is found to be equal to the reciprocal of the deflection at the effective height Δ_e, as follows

$$K = \frac{1}{\Delta_e}$$

This model, although not strictly correct, can be used to determine the elastic distribution of the torsional forces as well as the displacements, as illustrated in Example 2 in Chapter 4.

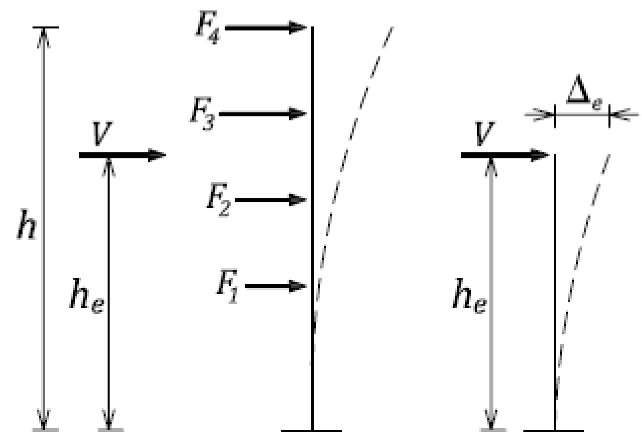


Figure C-17. Vertical combination of wall segments with different stiffness properties.

Several different elastic analysis approaches can be used to determine the stiffness of a wall with openings. A simplified approach suitable for the stiffness calculation of a perforated wall in a single-storey building can be explained with the help of an example of the wall X_1 shown in Figure C-18 (see also Example 3 in Chapter 4). For a unit load applied at the top, the wall stiffness calculation involves the following steps:

- First, calculate the deflection at the top for a cantilever wall, considering the wall to be solid (Δ_{solid}).
- Next, calculate the deflection for the strip containing openings (Δ_{strip}), considering the full wall length (i.e. ignore openings).
- Finally, calculate the deflection for the piers A, B, C, and D (Δ_{ABCD}) assuming that all piers have the same deflection.

Note that the deflections for individual components are calculated as the inverse of their stiffness values, and that the pier stiffnesses are determined assuming either the cantilever or fixed-end models. In most cases, the use of the cantilever model is more appropriate.

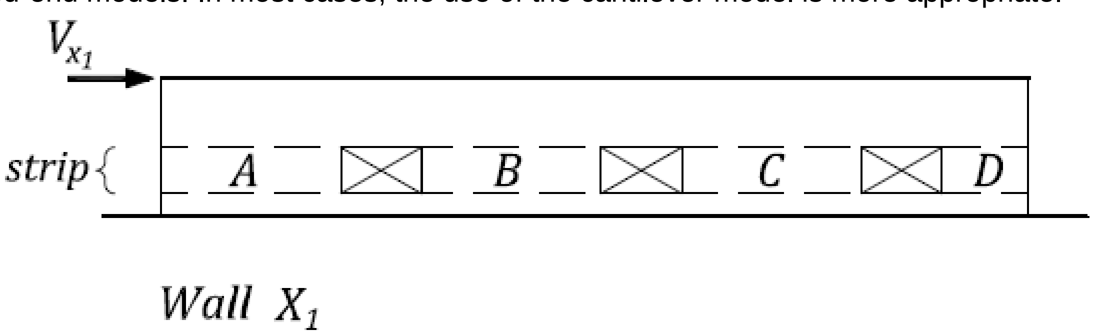


Figure C-18. An example of a perforated wall.

The overall wall deflection can be determined by combining the deflections for these components, as follows:

$$\Delta = \Delta_{solid} - \Delta_{strip} + \Delta_{ABCD}$$

Note that the strip deflection is subtracted from the solid wall deflections - this removes the entire portion of the wall containing all the openings, which is then replaced by the four segments.

Finally, the wall stiffness is equal to the reciprocal of the deflection, as follows

$$K = \frac{1}{\Delta}$$

C.3.4 Advanced Design Approaches for Reinforced Masonry Shear Walls with Openings

The approximate approach based on elastic analysis presented in Section C.3.3 is appropriate for determining the lateral force distribution in masonry walls. However, that method is not adequate for predicting the strengths in perforated reinforced masonry shear walls (walls with openings). Openings in a masonry shear wall alter its behaviour and add complexity to its analysis and design. When the openings are relatively small, their effect can be ignored, however in most walls the openings need to be considered. The following two design approaches can be used to design walls with openings:

1) Plastic analysis method, and
2) Strut-and-tie method.

These two approaches have been evaluated by experimental studies and have shown very good agreement with the experimental results (Voon, 2007; Elshafie et al., 2002; Leiva and Klingner, 1994). The key concepts will be outlined in this section.

C.3.4.1 Plastic analysis method

The plastic analysis method, also known as limit analysis, can be used to determine the ultimate load-resisting capacity for statically indeterminate structures. A masonry wall with an opening as shown in Figure C-19a can be modeled as a frame (see Figure C-19b). The model is subjected to an increasing load until the flexural capacity of a specific section is reached and a *plastic hinge* is formed at that location. (The plastic hinge is a region in the member that is assumed to be able to undergo an infinite amount of deformation, and can therefore be treated as a hinge for further analysis.) With further load increases, plastic hinges will be formed at other sections as their flexural capacity is reached. This process continues until the system becomes statically determinate, at which point the formation of one more plastic hinge will result in a collapse under any additional load. This is called a collapse mechanism, and an example is shown in Figure C-19c. There is usually more than one possible collapse mechanism for a statically indeterminate structure, and the mechanism that gives the lowest capacity is closest to the ultimate capacity, as this is an upper bound method.

For specific application to perforated masonry walls, the wall is idealized as an equivalent frame, where piers are modeled as fixed at the base and either pinned or fixed at the top, while lintels are modeled as fixed at the ends. A failure state is reached when plastic hinges form at member ends, and the collapse mechanism forms. The sequence of plastic hinge formation depends on the relative strength and stiffness of the elements. In this approach, structural members must be designed to behave mainly in a flexural mode, while a shear failure is avoided by applying the capacity design approach.

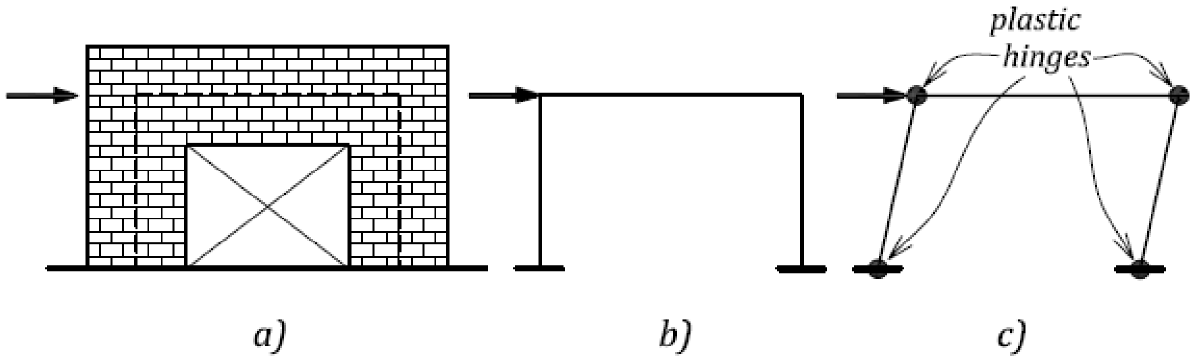
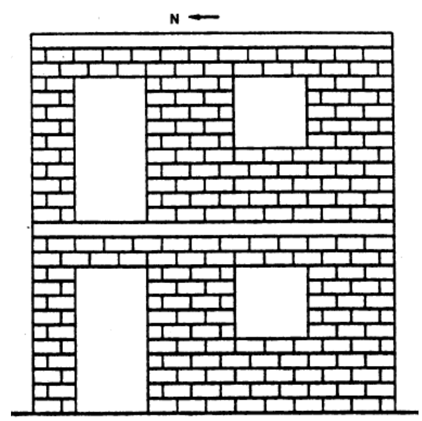


Figure C-19. An example of a plastic collapse mechanism for a frame system: a) perforated masonry wall; b) frame model; c) plastic collapse mechanism.

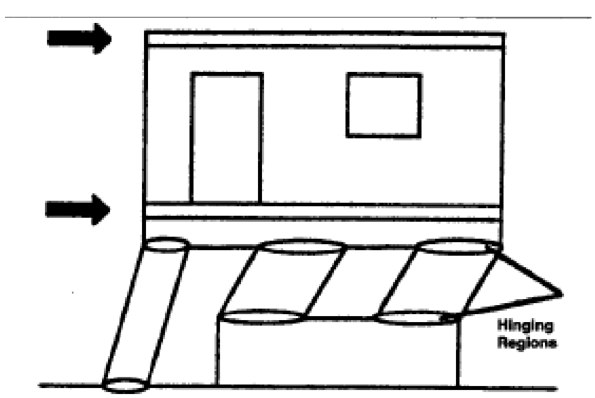
The following two mechanisms are considered appropriate for the plastic analysis of reinforced masonry walls with openings, as shown in Figure C-20 (Leiva and Klingner, 1994; Leiva et al. 1990):

1) pier mechanism, and

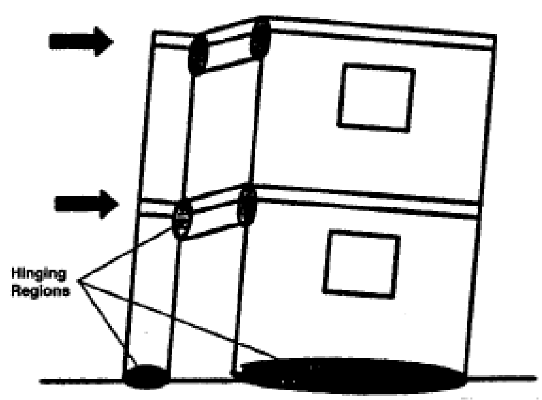
2) coupled wall mechanism.



a)



b)



c)

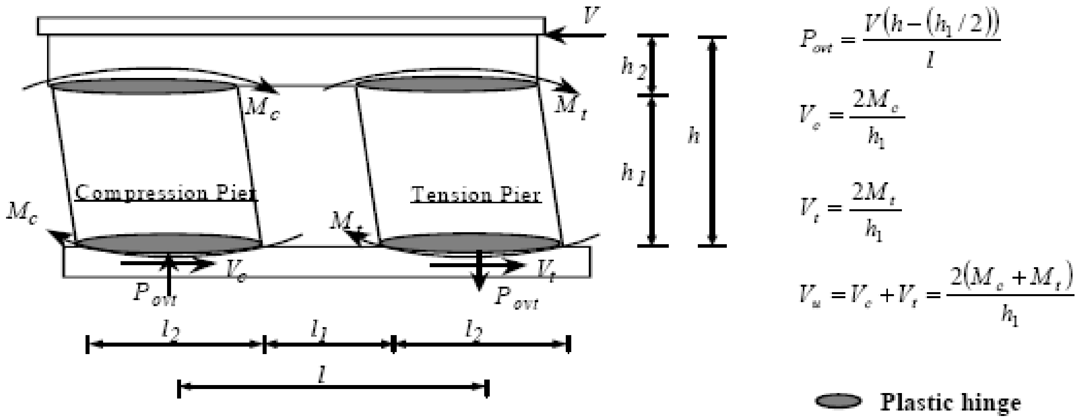
Figure C-20. Plastic analysis models for perforated walls: a) actual wall; b) pier model; c) coupled wall model (Leiva and Klingner, 1994, reproduced by permission of the Masonry Society).

A pier mechanism is a collapse mechanism with flexural hinges at tops and bottoms of the piers. A pier-based design philosophy visualizes a perforated wall as a ductile frame. Horizontal reinforcement above and below the openings is needed to transfer the pier shears into the rest of the wall. A drawback of the pier mechanism is that the formation of plastic hinges at the top and bottom of all piers at a story level can lead to significant damage to the piers, which are the main vertical load-carrying elements.

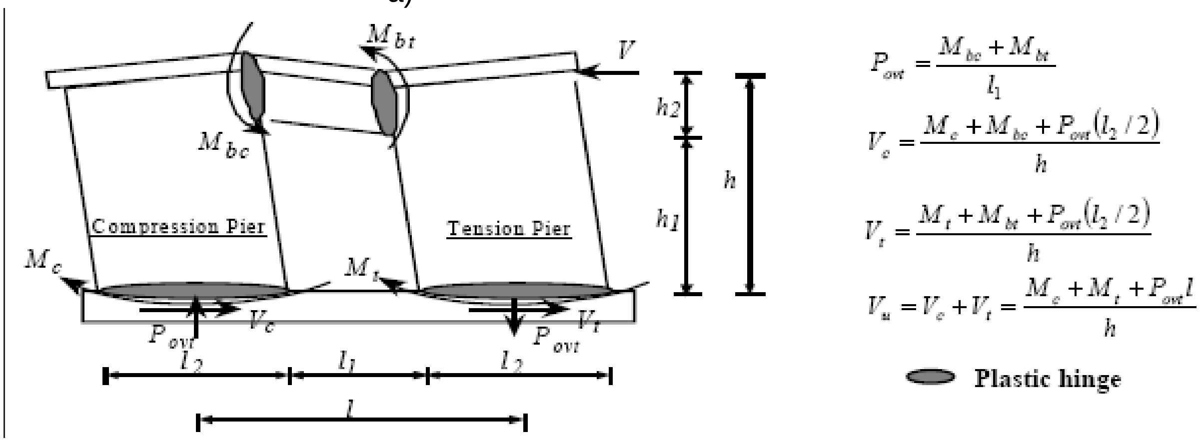
A coupled wall mechanism is a collapse mechanism in which flexural hinges are formed at the base of the wall and at the ends of the coupling lintels. A perforated wall is modeled as a series of ductile coupled walls; this concept is similar to that used for seismic design of reinforced concrete shear walls. The vertical reinforcement in each pier must be designed so that the flexural capacity of the piers exceeds the flexural capacity of the coupling beams. To achieve this, additional longitudinal reinforcement is placed in the piers, but cut off before it reaches the wall base. The shear reinforcement in the coupling beams is designed based on the flexural and shear capacity of the piers. Since masonry walls are usually long in plan, the formation of plastic hinges at their bases produces large strains in the wall longitudinal reinforcement. Plastic hinges must have adequate rotational capacity to allow the complete mechanism to form; this can be achieved in wall structures with low axial load. To ensure the successful application of the plastic analysis method, the wall reinforcement must be detailed to develop the necessary strength and inelastic deformation capacity.

Figure C-21 shows a simple single-storey wall that is analyzed for the two mechanisms. Ultimate shear forces corresponding to the pier and coupled wall mechanisms can be determined from the equations of equilibrium assuming that the moments at the plastic hinge locations are known. These equations are summarized in Figure C-21 (Elshafaie et al., 2002).

The plastic analysis method has a few advantages: stiffness calculations are not required, and the designer can choose the failure mechanism which ensures a desirable ductile response. The designer needs to have a general background in plastic analysis, which is covered in several references, e.g. Bruneau, Uang, and Whittaker (1998) and Ferguson, Breen, and Jirsa (1988). This method is also used for the seismic analysis of concrete and steel structures, and is referred to as nonlinear static analysis or pushover analysis.



a)



b)

Figure C-21. Ultimate wall forces according to the plastic analysis method: a) pier mechanism; b) coupled wall mechanism (Elshafaie et al., 2002, reproduced by permission of the Masonry Society).

C.3.4.2 Strut-and-Tie Method

The strut-and-tie method essentially follows the truss analogy approach used for shear design of concrete and masonry structures. Pin-connected trusses consist of steel tension members (ties), and masonry compression members (struts). The masonry compression struts develop between parallel inclined cracks in the regions of high shear. The essential feature of this approach is that the designer needs to find a system of internal forces that is in equilibrium with the externally applied loads and support conditions. A further essential feature is that the designer must ensure that the steel and masonry tie members provided adequately resist the forces obtained from the truss analysis.

The design of tension ties is particularly important. If a ductile response is to be assured, the designer should choose particular tension chords in which yielding can best be accommodated. Other ties can be designed so that no yielding will occur by using the capacity design approach. The magnitudes of the forces in critical tension ties can be determined from statics, corresponding to the overturning moment capacity of the wall using the nominal material properties (rather than the factored ones). The remaining forces are then determined from the equilibrium of nodes (conventional truss analysis). Compression forces developed in masonry struts are usually small due to the small compression strains and do not govern the design.

Careful detailing of the wall reinforcement is necessary to ensure that the actual structural response will correspond to that predicted by the analytical model.

The designer needs to use judgement to simplify the force paths that are chosen to represent the real structure – these differ considerably depending on individual judgement.

An example of a strut-and-tie model for a two-storey perforated masonry wall subjected to seismic lateral load is shown in Figure C-22 (note that gravity load also needs to be considered in the analysis, however it is omitted from the figure). It can be seen that two different models are required to account for the alternate direction of seismic load. The examples show the seismic load being applied as a compressive load to the building; however, these loads should be applied to the floor levels, depending on the diaphragm-to-wall connection. The designated tie members in one model will become struts in the other model (when the seismic load changes direction). An advantage of the reversible nature of seismic forces is that a significant fraction of the inelastic tensile strains imposed on the end strut members is recoverable due to force reversal, thereby providing hysteretic energy dissipation. A detailed solution for this example is presented in the User's Guide by NZCMA (2004).

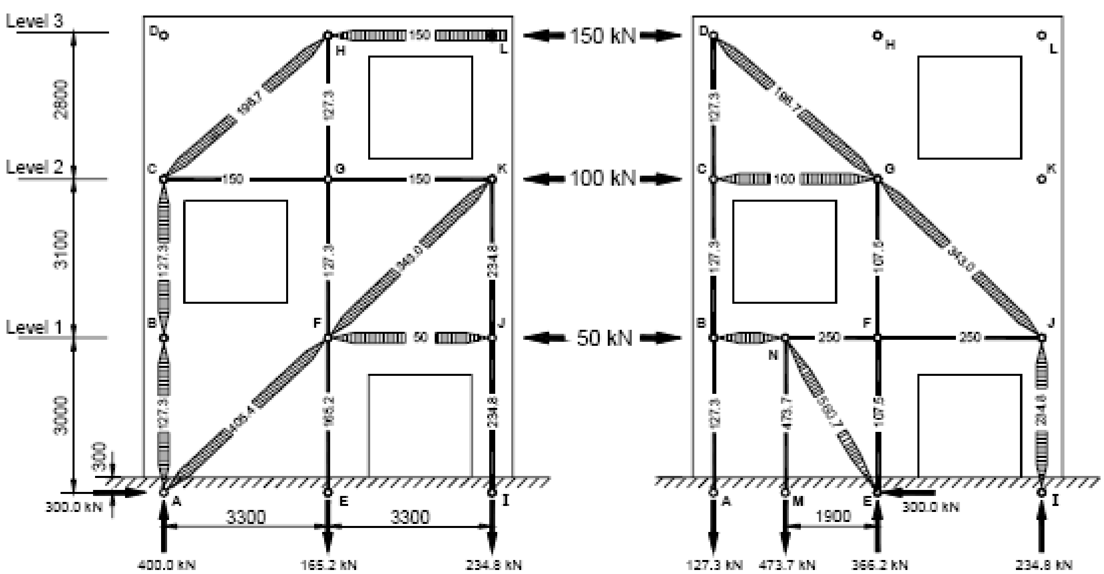


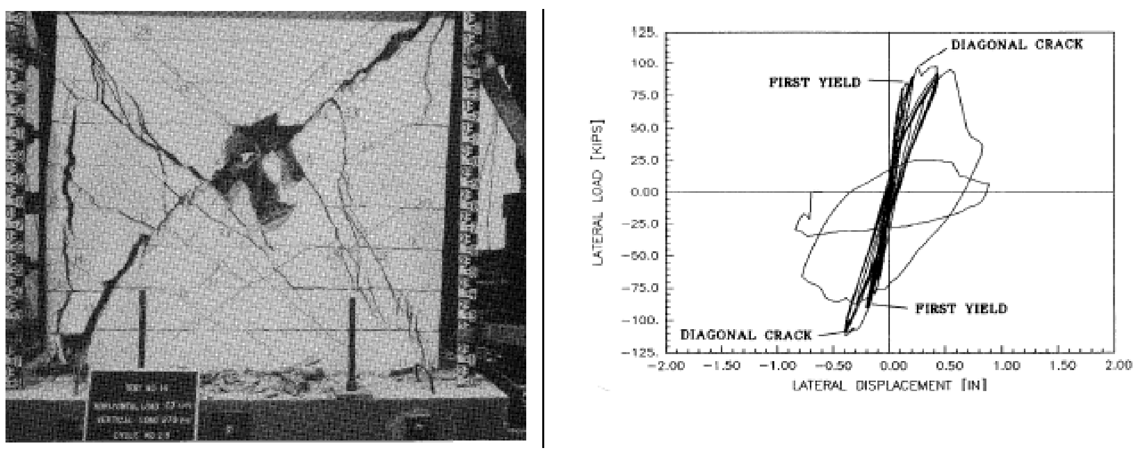
Figure C-22. Strut-and-tie models for a masonry wall corresponding to different directions of seismic loading (NZCMA, 2004, reproduced by the permission of the New Zealand Concrete Masonry Association Inc.).

Strut-and-tie models are used for design of masonry walls in New Zealand, and this approach is explained in more detail by Paulay and Priestley (1992). The New Zealand Masonry Standard NZS 4230:2004 (SANZ, 2004) recommends the use of strut-and-tie models for the design of perforated reinforced masonry shear walls. In Canada, strut-and-tie models are used to design discontinuous regions of reinforced concrete structures according to the Standard CSA A23.3-04 Design of Concrete Structures. The design concepts and applications of strut-and-tie models for concrete structures in Canada are covered by McGregor and Bartlett (2000).

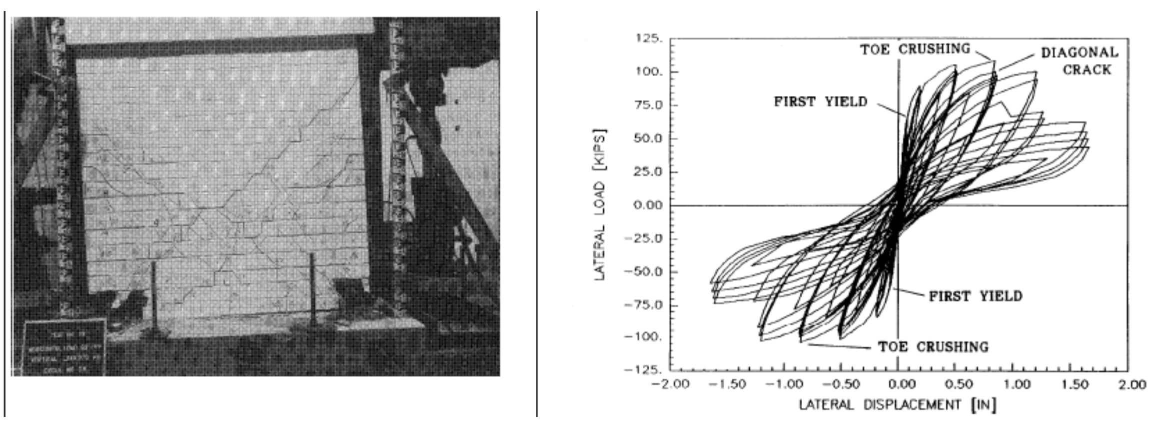
C.3.5 The Effect of Cracking on Wall Stiffness

The behaviour of masonry walls under seismic load conditions is rather complex, and depends on the failure mechanism (shear-dominant or flexure-dominant), as discussed in Section 2.3.1. Figure C-23 shows the hysteretic response of shear-dominant and flexure-dominant walls. The effective stiffness discussed in this section reflects the secant stiffness up to first crack in the brittle shear-dominant walls, and the stiffness for an elastic-perfectly-plastic model that would approximate the strength envelope of the hysteretic plot in the ductile flexure-dominant walls.

For the *shear-dominant mechanism*, the response is initially elastic until cracking takes place, at which point there is a substantial drop in stiffness. This is particularly pronounced after the development of diagonal shear cracks. After a few major cracks develop, the load resistance is taken over by the diagonal strut mechanism, and the shear stiffness can be estimated by an appropriate strut model. However, the stiffness drops significantly shortly after the strut mechanism is formed, and can be considered to be zero for most practical purposes (see Figure C-23a). It is expected that an increase in the quantity of vertical and horizontal steel and/or the magnitude of axial compressive stress causes a reduced crack size and an increase in the shear stiffness (Shing et al., 1990).



a)



b)

Figure C-23. Cracking pattern and load-displacement curves for damaged masonry wall specimens: a) shear-dominant response, and b) flexure-dominant response (Shing et al., FEMA 307, reproduced by permission of the Federal Emergency Management Agency).

For the *flexure-dominant mechanism*, a drop in the stiffness immediately after the onset of cracking is not very significant. As can be seen from Figure C-23b, the stiffness drops after the yielding of vertical reinforcement takes place, and continues to drop with increasing inelastic lateral deformations (this depends on the ductility capacity of the wall under consideration). The specimen for which the results are shown in Figure C-23b showed yielding of vertical reinforcement and compressive crushing of masonry at the wall toes (Shing et al., 1989).

Note that the height of wall test specimens shown in Figure C-23 was 1.8 m (6 feet), thus a 2.5% drift ratio permitted by the NBCC 2005 for regular buildings corresponds to 1.8 inch displacement. It can be seen that the displacements and drift in these specimens are very low, particularly so for the shear-dominant specimen shown in Figure C-23a.

Evidence from studies that focus on quantifying the changes in in-plane wall stiffness under increasing lateral loading are limited, so CSA S304.1 and other masonry codes do not provide guidance related to this issue. Shing et al. (1990) tested a series of 22 cantilever block masonry wall specimens that were laterally loaded at the top, with a height/length aspect ratio of 1.0. Based on the experimental test data, they have recommended the following empirical equation for the lateral stiffness of a wall with a shear-dominant response

$$K_e = (0.2 + 0.1073 f_c) K_{shear} \le K_{el} \qquad (15)$$

where

$K_{shear} = \dfrac{E_m * t_e}{3 * \left(\dfrac{h}{l_w}\right)}$ is the shear stiffness of a wall/pier

h = wall height

l_w = wall length

t_e = effective wall thickness

f_c = axial compressive stress (MPa)

The above equation is based on the force/displacement measurements taken just after the first diagonal crack developed, in specimens with a height/length ratio of 1.0. For seismic applications where the walls are expected to yield in flexure before failing in shear, and the lateral stiffness is used to estimate the fundamental period of the structure and to determine the seismic displacements, it is more appropriate to determine the effective stiffness from a cracked section analysis at first yield of the tension reinforcement.

A study by Priestley and Hart (1989), based on the cracked transformed section stiffness at first yield of the tension reinforcement, recommends that the effective moment of inertia, I_e, of a wall can be approximated by:

$$I_e = \left(\frac{100}{f_y} + \frac{P_f}{f'_m A_e}\right) I_g \qquad (16)$$

where

f_y = steel yield strength (MPa)

P_f = factored axial load

A_e = effective cross-sectional area for the wall

f'_m = masonry compressive strength, and

$I_g = \frac{t_e * l_w^3}{12}$ is the gross moment of inertia of the wall.

Note that the first term in the bracket, $100/f_y$, is equal to 0.25 for $f_y = 400$ MPa (Grade 400 steel). The second term is a ratio of axial compressive stress in the wall, equal to P_f / A_e , and the masonry compressive strength, f'_m .

The above relation is based solely on consideration of flexural stiffness, and is a best fit relationship for several different values of height/length ratio (h/l_w), steel strength, vertical reinforcement ratio and axial load. Other considerations are whether the vertical reinforcement is uniformly distributed across the wall length or concentrated at the ends, and the effect of tension stiffening. The vertical reinforcement ratio is not included in the above expression, and as a result, the wall stiffness is overestimated for lightly reinforced walls and underestimated for heavily reinforced walls.

If it is assumed that wall cracking causes the same proportional decrease in the effective shear area as it does for the moment of inertia, then the stiffnesses can be combined to give the following equation for the reduced wall stiffness, K_{ce} ,

$$K_{ce} = (\frac{100}{f_y} + \frac{P_f}{f'_m A_e})K_c \qquad (17)$$

where

$$K_c = \frac{E_m * t_e}{\left(\frac{h}{l_w}\right)\left[4\left(\frac{h}{l_w}\right)^2 + 3\right]}$$

is the combined stiffness of an uncracked <u>cantilever</u> wall or pier, considering both the flexural and shear deformation components (refer to Section C.3.2 for the wall stiffness equations).

The terms in the large right hand bracket of the K_c equation give the comparative value of flexural deformation to shear deformation. At a h/l_w ratio of 1.0, flexure contributes 4/7 of the total deformation and shear 3/7, while at a h/l_w ratio of 0.5, shear contributes ¾ of the total defection.

The Priestley and Hart equation was obtained using experimental data related to cantilever wall specimens, however it may also be used for fixed-end walls. The stiffness equation for these walls, K_{fe} , is the same as for the cantilever walls, that is,

$$K_{fe} = (\frac{100}{f_y} + \frac{P_f}{f'_m A_e})K_f \qquad (18)$$

where

$K_f = \frac{E_m * t_e}{\left(\frac{h}{l_w}\right)\left[\left(\frac{h}{l_w}\right)^2 + 3\right]}$ is the stiffness of an uncracked <u>fixed-end</u> wall or a pier

A comparison of the proposed equations for a masonry block wall under axial compressive stress is presented in Figure C-24. The following values were used in the calculations: $f_y = 400$ MPa, P_f / A_e = 1 MPa, and f'_m = 10 MPa.

Note that the Shing equation is only shown for h/l_w up to 1.5 as it is based entirely on shear deformation. Since the Shing equation represents stiffness at first diagonal cracking, it is expected to give higher stiffness values than the Priestley-Hart equation. Use of the Priestley-Hart stiffness equation is recommended since it is valid for all h/l_w ratios.

The elastic uncracked stiffness could be used to distribute lateral seismic load to individual walls and piers, but the reduced cracked stiffness should be used for period estimation and deflection calculations.

The wall design deflections can be found from the following equation:

$$\Delta_{design} = \Delta_{el} * \frac{R_d * R_o}{I_E}$$

where

Δ_{el} = elastic deflections calculated using the reduced wall stiffness (K_{ce} or K_{fe}) and the factored design forces, and

$\frac{R_d * R_o}{I_E}$ = deflection multiplier to account for the effects of ductility, overstrength, and the building importance factor (see Section 1.5.11)

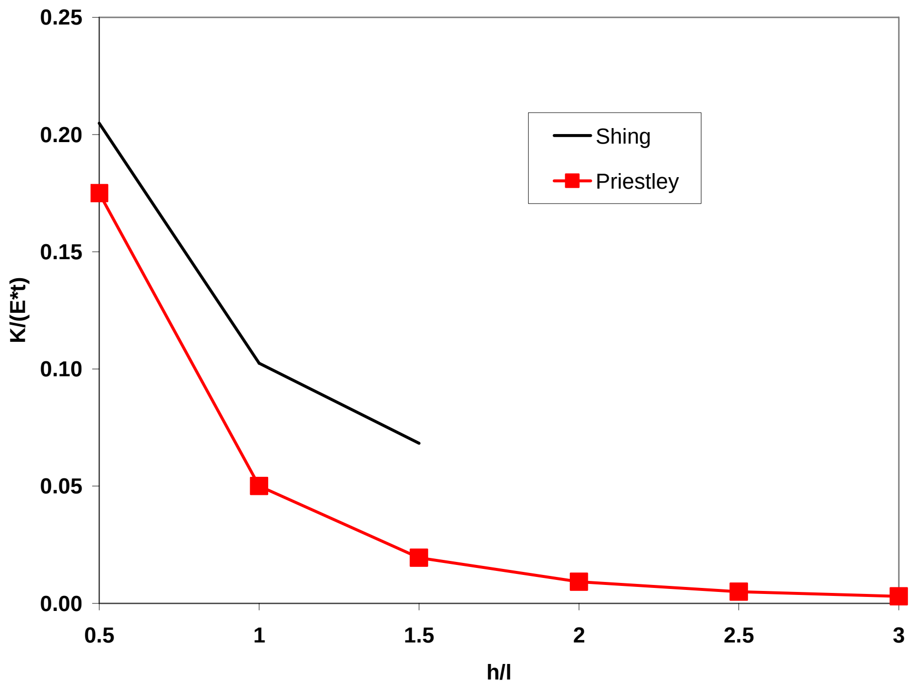


Figure C-24. A comparison of the stiffness values obtained using the Shing and Priestley-Hart equations.

TABLE OF CONTENTS – APPENDIX D

D Design Aids

Table D-1. Properties of Concrete Masonry Walls (per metre or foot length)[1]

Grouted Cells / metre		0.00	0.83	1.00	1.25	1.67	2.50	5.00
Cell/dowel Spacing (mm)		*none*	1200	1000	800	600	400	200
Nominal Size		***150***	mm		*6*	inch		
A_e	(mm^2 x 10^3)	52.0	66.7	69.6	74.0	81.3	96.0	140.0
	(in^2)	*24.6*	*31.5*	*32.9*	*35.0*	*38.4*	*45.4*	*66.2*
I_x	(mm^4 x 10^6)	172	181	183	186	191	201	229
	(in^4)	*126*	*133*	*134*	*136*	*140*	*147*	*168*
S_x	(mm^3 x 10^6)	2.46	2.59	2.62	2.66	2.73	2.87	3.27
	(in^3)	*45.8*	*48.2*	*48.7*	*49.5*	*50.7*	*53.3*	*60.8*
Weight	(kN/m^2)	1.90	2.09	2.13	2.19	2.29	2.49	3.08
	(psf)	*39.6*	*43.7*	*44.6*	*45.8*	*47.9*	*52.0*	*64.3*
Nominal Size		***200***	mm		*8*	inch		
A_e	(mm^2 x 10^3)	75.4	94.5	98.3	104.0	113.6	132.7	190.0
	(in^2)	*35.6*	*44.6*	*46.5*	*49.2*	*53.7*	*62.7*	*89.8*
I_x	(mm^4 x 10^6)	442	464	468	475	485	507	572
	(in^4)	*324*	*340*	*343*	*347*	*355*	*371*	*419*
S_x	(mm^3 x 10^6)	4.66	4.88	4.93	5.00	5.11	5.34	6.02
	(in^3)	*86.7*	*90.9*	*91.7*	*93.0*	*95.0*	*99.3*	*112.0*
Weight	(kN/m^2)	2.46	2.75	2.81	2.89	3.03	3.32	4.18
	(psf)	*51.4*	*57.4*	*58.6*	*60.4*	*63.4*	*69.4*	*87.3*
Nominal Size		***250***	mm		*10*	inch		
A_e	(mm^2 x 10^3)	81.7	108.1	113.4	121.3	134.5	160.9	240.0
	(in^2)	*38.6*	*51.1*	*53.6*	*57.3*	*63.5*	*76.0*	*113.4*
I_x	(mm^4 x 10^6)	816	872	883	900	928	984	1152
	(in^4)	*598*	*638*	*647*	*659*	*679*	*721*	*844*
S_x	(mm^3 x 10^6)	6.80	7.27	7.36	7.50	7.73	8.20	9.60
	(in^3)	*126.5*	*135.2*	*136.9*	*139.5*	*143.8*	*152.5*	*178.6*
Weight	(kN/m^2)	2.97	3.35	3.43	3.55	3.74	4.12	5.28
	(psf)	*62.0*	*70.0*	*71.7*	*74.1*	*78.1*	*86.1*	*110.3*
Nominal Size		***300***	mm		*12*	inch		
A_e	(mm^2 x 10^3)	88.3	121.9	128.6	138.7	155.5	189.2	290.0
	(in^2)	*41.7*	*57.6*	*60.8*	*65.5*	*73.5*	*89.4*	*137.0*
I_x	(mm^4 x 10^6)	1341	1456	1479	1514	1571	1687	2032
	(in^4)	*982*	*1066*	*1083*	*1108*	*1150*	*1235*	*1488*
S_x	(mm^3 x 10^6)	9.25	10.04	10.20	10.44	10.83	11.63	14.01
	(in^3)	*172.1*	*186.8*	*189.7*	*194.1*	*201.5*	*216.3*	*260.6*
Weight	(kN/m^2)	3.53	4.00	4.10	4.24	4.48	4.95	6.38
	(psf)	*73.7*	*83.6*	*85.6*	*88.6*	*93.6*	*103.5*	*133.3*
Note:	Assume Bond Beams at 2.4 m (8 ft) O.C.							
	Table based on Metric blocks and modules (190 mm high units)							
	Assumed Weight	22	kN/m3	140.4	pcf			

[1] Source: Masonry Technical Manual (MIBC, 2008, reproduced by permission of the Masonry Institute of BC)

Table D-2. c/l_w *ratio,* f_y *= 400 MPa*

ω	α										
	0.000	0.025	0.050	0.075	0.100	0.150	0.200	0.250	0.300	0.350	0.400
0	0.000	0.037	0.074	0.110	0.147	0.221	0.294	0.368	0.441	0.515	0.588
0.01	0.014	0.050	0.086	0.121	0.157	0.229	0.300	0.371	0.443	0.514	0.586
0.02	0.028	0.063	0.097	0.132	0.167	0.236	0.306	0.375	0.444	0.514	0.583
0.03	0.041	0.074	0.108	0.142	0.176	0.243	0.311	0.378	0.446	0.514	0.581
0.04	0.053	0.086	0.118	0.151	0.184	0.250	0.316	0.382	0.447	0.513	0.579
0.05	0.064	0.096	0.128	0.160	0.192	0.256	0.321	0.385	0.449	0.513	0.577
0.06	0.075	0.106	0.138	0.169	0.200	0.263	0.325	0.388	0.450	0.513	0.575
0.07	0.085	0.116	0.146	0.177	0.207	0.268	0.329	0.390	0.451	0.512	0.573
0.08	0.095	0.125	0.155	0.185	0.214	0.274	0.333	0.393	0.452	0.512	0.571
0.09	0.105	0.134	0.163	0.192	0.221	0.279	0.337	0.395	0.453	0.512	0.570
0.1	0.114	0.142	0.170	0.199	0.227	0.284	0.341	0.398	0.455	0.511	0.568
0.11	0.122	0.150	0.178	0.206	0.233	0.289	0.344	0.400	0.456	0.511	0.567
0.12	0.130	0.158	0.185	0.212	0.239	0.293	0.348	0.402	0.457	0.511	0.565
0.13	0.138	0.165	0.191	0.218	0.245	0.298	0.351	0.404	0.457	0.511	0.564
0.14	0.146	0.172	0.198	0.224	0.250	0.302	0.354	0.406	0.458	0.510	0.563
0.15	0.153	0.179	0.204	0.230	0.255	0.306	0.357	0.408	0.459	0.510	0.561
0.16	0.160	0.185	0.210	0.235	0.260	0.310	0.360	0.410	0.460	0.510	0.560
0.17	0.167	0.191	0.216	0.240	0.265	0.314	0.363	0.412	0.461	0.510	0.559
0.18	0.173	0.197	0.221	0.245	0.269	0.317	0.365	0.413	0.462	0.510	0.558
0.19	0.179	0.203	0.226	0.250	0.274	0.321	0.368	0.415	0.462	0.509	0.557
0.2	0.185	0.208	0.231	0.255	0.278	0.324	0.370	0.417	0.463	0.509	0.556

Input parameters:

$$\rho_{vflex} = \frac{A_{vt}}{t * l_w}$$

$$\omega = \frac{566.7 * \rho_{vflex}}{f'_m}$$

$$\alpha = \frac{1667 * P_f}{f'_m l_w t}$$

Units:

P_f (kN)

l_w, t (mm)

A_{vt} (mm^2)

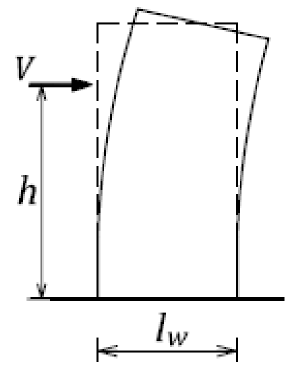
f'_m (MPa)

Table D-3. Wall Stiffness Values $K/(E_m * t)$

h / l	Cantilever	Fixed
0.05	6.645	6.661
0.1	3.289	3.322
0.15	2.157	2.206
0.2	1.582	1.645
0.25	1.231	1.306
0.3	0.992	1.079
0.35	0.819	0.915
0.4	0.687	0.791
0.45	0.583	0.694
0.5	0.500	0.615
0.55	0.432	0.551
0.6	0.375	0.496
0.65	0.328	0.450
0.7	0.288	0.409
0.75	0.254	0.374
0.8	0.225	0.343
0.85	0.200	0.316
0.9	0.178	0.292
0.95	0.159	0.270
1	0.143	0.250
1.05	0.129	0.232
1.1	0.116	0.216
1.15	0.105	0.201
1.2	0.095	0.188
1.25	0.086	0.175
1.3	0.079	0.164
1.35	0.072	0.154
1.4	0.066	0.144
1.45	0.060	0.135
1.5	0.056	0.127
1.55	0.051	0.119
1.6	0.047	0.112
1.65	0.044	0.106
1.7	0.040	0.100
1.75	0.037	0.094
1.8	0.035	0.089
1.85	0.032	0.084
1.9	0.030	0.080
1.95	0.028	0.075
2	0.026	0.071

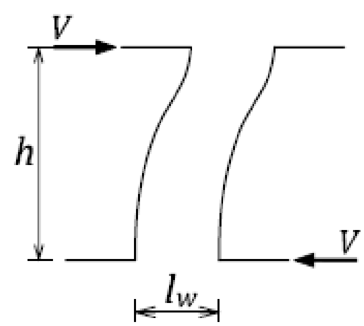
Cantilever model:

$$\frac{K}{E_m * t} = \frac{1}{\left(\frac{h}{l_w}\right)\left[4\left(\frac{h}{l_w}\right)^2 + 3\right]}$$



Fixed both ends:

$$\frac{K}{E_m * t} = \frac{1}{\left(\frac{h}{l_w}\right)\left[\left(\frac{h}{l_w}\right)^2 + 3\right]}$$



$E_m = 850 f'_m$ Modulus of elasticity

$G = 0.4E_m$ Modulus of rigidity (shear modulus)

$A_v = 5A/6$ Shear area

E Notation

$a_{\max}$ = maximum acceleration

a = depth of the compression zone (equivalent rectangular stress block)

a_w = clear distance between the adjacent cross walls

A_b = area of reinforcement bar

A_c = area of concentrated reinforcement at each end of the wall

A_d = area of distributed reinforcement along the wall length

A_e = effective cross-sectional area of masonry

A_g = gross cross-sectional area of masonry

A_L = area of the compression zone (flanged wall section)

A_r = response amplification factor to account for the type of attachment of equipment or veneer ties

A_{uc} = uncracked area of the cross-section

A_v = area of horizontal wall reinforcement

A_{vt} = total area of the distributed vertical reinforcement

A_v = shear area of the wall section

A_x = amplification factor at level x to account for variation of response with the height of the building (veneer tie design)

b = effective width of the compression zone

b_{actual} = actual flange width

b_c = critical wall thickness

b_T = overhanging flange width

b_w = overall web width (shear design)

B = torsional sensitivity factor

c = neutral axis depth (distance from the extreme compression fibre to the neutral axis)

C = compressive force in the masonry acting normal to the sliding plane

C_m = the resultant compression force in masonry

C_h = compressive force in the masonry acting normal to the head joint

C_p = seismic coefficient for a nonstructural component (veneer tie design)

d = effective depth (distance from the extreme compression fibre to centroid of tension reinforcement)

d_v = effective wall depth for shear calculations

d' = distance from the extreme compression fibre to the centroid of the concentrated compression reinforcement

D_{nx} = plan dimension of the building at level x perpendicular to the direction of seismic loading being considered

e = load eccentricity

e_a = accidental torsional eccentricity

e_x = torsional eccentricity (distance measured perpendicular to the direction of earthquake loading between the centre of mass and the centre of rigidity at the level being considered)

E_f = modulus of elasticity of the frame material (infill walls)

E_m = modulus of elasticity of masonry

f_t = flexural tensile strength of masonry (see Table 5 of CSA S304.1-04)

f'_m = compressive strength of masonry normal to bed joints at 28 days (see Table 4 of CSA S304.1-04)

f_y = yield strength of reinforcement

F = force

F_t = a portion of the base shear V applied at the top of the building

F_{el} = elastic force

F_a = acceleration-based site coefficient

F_v = velocity-based site coefficient

F_x = seismic force applied to level x

F_y = yield force

G = modulus of rigidity for masonry (shear modulus)

h = unsupported wall height/height of the infill wall

h_w = total wall height

h_n = building height

h_s = storey height

h_x = height from the base of the structure up to the level x

I_b = moment of inertia of the beam

I_c = moment of inertia of the column

I_E = earthquake importance factor of the structure

J = numerical reduction coefficient for base overturning moment

k = effective length factor for compression member

K = stiffness

l = length of the infill wall

l_d = design length of the diagonal strut (infill wall)

l_p = plastic hinge length

l_s = length of the diagonal strut

l_w = wall length

L_n = clear vertical distance between lines of effective horizontal support or clear horizontal distance between lines of effective vertical support

M = mass

M_f = factored bending moment

M_r = factored moment resistance

M_n = nominal moment resistance

M_p = probable moment resistance

M_v = factor to account for higher mode effect on base shear

N = axial load arising from bending in coupling beams or piers

p_f = distributed axial stress

P_d = axial compressive load on the section under consideration

P_{cr} = critical axial compressive load

P_{DL} = dead load

P_{fb} = the resultant compression force (flanged walls)

P_r = factored axial load resistance

P_1 = compressive force in the unreinforced masonry acting normal to the sliding plane

P_2 = compressive force in the reinforced masonry acting normal to the sliding plane

P_h = horizontal component of the diagonal strut compression resistance (infill walls)

P_v = the vertical component of the diagonal strut compression resistance (infill walls)

P_{ult} = ultimate tie strength

R_d = ductility-related force modification factor

R_o = overstrength-related force modification factor

R_p = element or component response modification factor (veneer tie design)

s = reinforcement spacing

$S(T)$ = design spectral acceleration

$S_a(T)$ = 5% damped spectral response acceleration

S_e = section modulus of effective wall cross-sectional area

S_p = horizontal force factor for part or portion of a building and its anchorage (veneer tie design)

t = overall wall thickness

t_e = effective wall thickness

t_f = face shell thickness

T = fundamental period of vibration of the building

T_x = torsional moment at level x

T_r = the resultant force in steel reinforcement

T_y = factored tensile force at yield of the vertical reinforcement

v_f = distributed shear stress

v_m = masonry shear strength

$v_{\max}$ = maximum velocity

V = lateral earthquake design force at the base of the structure

V_e = lateral earthquake elastic force at the base of the structure

V_f = factored shear force

V_{nb} = the resultant shear force corresponding to the development of nominal moment resistance M_n at the base of the wall

V_m = masonry shear resistance

V_r = factored shear resistance

$\overline{V}_s$ = average shear wave velocity in the top 30 m of soil or rock

V_s = factored shear resistance of steel reinforcement

w = diagonal strut width (infill walls)

w_e = effective diagonal strut width (infill walls)

W = seismic weight, equal to the dead weight plus some portion of live load that would move laterally with the structure

W_p = weight of a part or a portion of a structure (veneer tie design)

W_x = a portion of seismic weight W that is assigned to level x

α_h = vertical contact length between the frame and the diagonal strut (infill walls)

α_L = horizontal contact length between the frame and the diagonal strut (infill walls)

β = damping ratio

β_d = ratio of the factored dead load moment to the total factored moment

β_1 = ratio of depth of rectangular compression block to depth of the neutral axis

γ_g = factor to account for partially grouted or ungrouted walls that are constructed of hollow or semi-solid units

$\delta_{\max}$ = the maximum storey displacement at level x at one of the extreme corners in the direction of earthquake

δ_{ave} = the average storey displacement determined by averaging the maximum and minimum displacements of the storey at level x

Δ = lateral displacement

Δ_p = plastic displacement

Δ_y = displacement at the onset of yielding

Δ_{el} = elastic displacement

$\Delta_{\max}$ = maximum displacement

Δ_u = inelastic (plastic) displacement

ε_m = the maximum compressive strain in masonry

ε_s = strain in steel reinforcement

ε_y = yield strain in steel reinforcement

χ = factor used to account for direction of compressive stress in a masonry member relative to the direction used for determination of f'_m

φ = curvature

φ_u = ultimate curvature

φ_y = yield curvature corresponds to the onset of yielding

ϕ_{er} = resistance factor for member stiffness

ϕ_m = resistance factor for masonry

ϕ_s = resistance factor for steel reinforcement

ϕ = resistance factor

ρ_v = vertical reinforcement ratio

ρ_h = horizontal reinforcement ratio

μ = coefficient of friction

μ_Δ = displacement ductility ratio (Chapter 1)

μ_φ = curvature ductility ratio

μ_Δ = displacement ductility ratio

θ = angle of diagonal strut measured from the horizontal

θ_e = elastic rotation

θ_p = plastic rotation

ω = natural frequency