EXPERIMENTAL AND NUMERICAL INVESTIGATIONS OF DUCTILE SLENDER REINFORCED MASONRY SHEAR WALLS SUBJECTED TO IN-PLANE SEISMIC LOADS

by

Brook Raymond Robazza

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B.A.Sc., The University of British Columbia, 2010

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The following individuals certify that they have read, and recommend to the Faculty of Graduate and Postdoctoral Studies for acceptance, the dissertation entitled:

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submitted by Brook Raymond Robazza in partial fulfillment of the requirements for
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in Civil Engineering

Examining Committee:

Tony Y. Yang
Supervisor

Svetlana Brzev
Supervisory Committee Member

Donald L. Anderson
Supervisory Committee Member

Carlos E. Ventura
University Examiner

Frank C.F. Lam
University Examiner
Abstract

Ductile slender reinforced masonry shear walls (DSRMSWs), defined here as ductile walls with height-to-thickness (h_u/t_w) ratios greater than 20 and height-to-length (h_u/L_w) ratios greater than 1.5, that are designed and detailed with modern seismic design provisions are often used as the seismic-force-resisting system (SFRS) for contemporary buildings. The in-plane seismic performance of these walls is however relatively poorly understood compared to other types of SFRS shear walls, particularly with regards to their lateral stability during in-plane seismic loading. This is partially because the majority of recent experimental testing on reinforced masonry shear walls (RMSWs) has been conducted using wall specimens that are either non-slender, with design parameters that do not reflect walls typically used in current Canadian masonry construction practice, or that do not experience any form of lateral instability. Moreover, as the Canadian Standards Association standards transition to performance-based design provisions, there becomes a need for practical and reliable numerical models that have been developed and validated using experimental results, which are limited at this time.

This dissertation presents experimental and analytical studies examining the in-plane performance of DSRMSWs undergoing simulated seismic effects. The experimental phase involved the testing of eight full-scale DSRMSWs (two of which were tested during the author’s M.A.Sc. thesis work) composed of fully-grouted concrete block masonry units with varying h_u/L_w and h_u/t_w ratios, amount and distribution of reinforcement, cross-sectional shape, axial stress level, and type of cyclic loading protocol.

The analytical phase first analyzed the results of the experimental phase to classify and improve the understanding of the failure modes affecting DSRMSWs, as well as to compare
current design provisions of several international masonry design codes. The analytical phase also employed a nonlinear multiple-vertical-line-element (MVLE) model that was calibrated using the numerical results of the specimens tested in the experimental phase. It was demonstrated that the model was able to reproduce the observed in-plane lateral load-displacement responses of both the experimental phase specimens as well as those of another study by others, with reasonable accuracy. The proposed MVLE model may be used as a useful tool for practicing engineers following performance-based design provisions for DSRMSWs.
Lay Summary

Ductile slender reinforced masonry shear walls (DSRMSWs) that are designed using the Canadian masonry design code are often used to provide the seismic support for contemporary buildings in Canada and other countries. The ability for these walls to resist seismic shaking is however not well understood compared to other types of shear walls. This is partially because the majority of recent laboratory testing on reinforced masonry shear walls has been conducted using specimens that are non-slender or do not reflect walls that are typically used in current Canadian masonry construction practice. Moreover, as the Canadian masonry design codes continue to modernize, engineers need practical analysis tools to design structures with DSRMSWs. This dissertation describes an experimental program studying the in-plane performance of practical DSRMSWs tested in a laboratory, compares current design codes, and provides recommendation for a convenient analysis tool for practicing engineers to use when designing DSRMSWs.
Preface

A significant portion of the content in this dissertation has been modified or adapted from parts of journal papers and conference proceedings that have been prepared and presented by the author, Brook Robazza, as described below. In all of these journal papers and conference proceedings, I was the primary author of all of the work, with the coauthors and collaborators providing technical guidance and editorial comments.


This dissertation is based on original work by me. The concept and design of the test specimens and test setup is my own. I carried out the majority of the assembly and construction of the test setups, most of the material testing, and directed all of the full-scale testing and data recording. The set-up of the analytical model in this study, including all code and data processing, is my work.
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List of Symbols

\( A_{env} \) = area below the experimental resistance envelope;
\( A_{m/c} \) = area of end zone in RMSWs/RCSWs;
\( A_{nv} \) = net cross-sectional area of the member subjected to considered shear demand, \( \text{mm}^2 \);
\( A_{ps} \) = cross-sectional area of prestressed reinforcement in flexural tension zone, \( \text{mm}^2 \);
\( A_{reqd} \) = cross-sectional area of flexural tensile reinforcement required to maintain axial equilibrium, \( \text{mm}^2 \);
\( A_s \) = area of vertical reinforcement within the wall end zone;
\( A_{tr} \) = area of transverse reinforcing bars, \( \text{mm}^2 \);
\( b_w \) = effective wall web width, mm;
\( c \) = the distance from the maximum compressive fibre to the neutral axis, mm;
\( C \) = compressive force in the masonry acting normal to the sliding plane, normally taken as \( P_d \) plus the factored tensile force at yield of the vertical reinforcement that is detailed to develop yield strength on both sides of the sliding plane;
\( d \) = distance from extreme compression fibre to centroid of longitudinal tension reinforcement, but need not be less than 0.8 times the horizontal length of the wall in the direction of the applied shear, or 0.9 times the overall depth of the member in the plan of the loading, mm;
\( d_l \) = diameter of longitudinal reinforcing bars, mm;
DSRMSW = Ductile Slender Reinforced Masonry Shear Wall;
\( d_t \) = diameter of reinforcing tie bars, mm;
\( d_u \) = storey drift at the point of maximum load;
\(d_{0.8u}\) = storey drift at the point of 20% strength degradation from the point of maximum load;

\(d_y\) = storey drift at first-yield of the outermost vertical reinforcement;

\(f_{bom}\) = design anchorage bond strength of reinforcing steel, MPa;

\(f_{bok}\) = characteristic anchorage bond strength of reinforcing steel, MPa;

\(f_d\) = design compressive strength of masonry, MPa = \(\frac{f_k}{\gamma_m}\);

\(f'_{gr}\) = grout compressive strength, MPa;

\(f_k\) = characteristic compressive strength of masonry in the direction of loading, MPa;

\(f'_m\) = masonry compressive strength, MPa;

\(f'_{mo}\) = mortar compressive strength, MPa;

\(f'_{mu}\) = masonry unit compressive strength, MPa;

\(f_s\) = calculated tensile or compressive stress in reinforcement, MPa;

\(f_{vd}\) = design shear strength of masonry, MPa;

\(f_{vk}\) = characteristic shear strength of masonry, MPa;

\(f_{vk0}\) = characteristic shear strength of masonry under zero compressive load, MPa;

\(f_{yd}\) = design yield strength of shear reinforcement, MPa;

\(f_{yk}\) = characteristic yield strength of reinforcing steel, MPa;

\(f_y\) = steel yield strength;

\(f_{yt}\) = specified yield strength of transverse reinforcement, MPa;

\(f'_{m'k}\) = masonry compressive strength (RMSWs) or concrete compressive strength (RCSWs);

\(h\) = wall height;

\(h_e\) = effective wall height in the plane of applied loading, mm;

\(h_{ef}\) = effective height of wall flange, mm;
$k_1 = (i) 1.3$ for horizontal reinforcement so placed that more than 300 mm of fresh grout is cast in the member below the development length or splice; (ii) $1.0$ for other cases;

$k_2 = (i) 1.5$ for epoxy-coated reinforcement with clear cover less than 3db, or with clear spacing between bars being developed less than 6db; (ii) $1.0$ for all other epoxy-coated reinforcement; and (iii) $1.0$ for uncoated reinforcement;

$k_3 = (i) 0.8$ for $db \leq 20$ mm; (ii) $1.0$ for $db > 20$ mm;

$K \leq$ (minimum masonry cover; clear spacing between adjacent reinforcement splices; 9db);

$K_e =$ effective wall stiffness of the EPP curve;

$K_y =$ effective wall stiffness to the onset of first-yield of the outermost vertical reinforcement;

$l_b =$ straight anchorage length for reinforcing bars with constant bond stress, mm

Where a greater area of reinforcing steel is provided than is required by design, the anchorage length may be reduced proportionally provided that reinforcing steel in tension is not less than the greater of 0.3lb, 10φ, or 100 mm

$L_{pt} =$ plastic hinge length, mm;

$L_v =$ wall shear span, mm;

$L_w =$ wall length;

$m =$ mechanical reinforcement ratio;

$MF =$ product of applicable modification factors: (a) $1.3$ for top horizontal reinforcing bars where more than 300 mm of fresh grout is cast in the component below the bar; (b) $1.0$ for reinforcing bars that do not contain an expansive admixture as defined in NZS 4210; (c) $Asr/Asp$ for reinforcing bars outside of critical sections of components subjected to earthquake loads;

$M_n =$ nominal moment strength, N-mm;

$M_u =$ factored moment demand (including second-order effects where applicable), N-mm;
\[ n = \text{numbers of bars being spliced or developed along the potential plane of bond splitting}; \]
\[ N^* = \text{design axial load in compression at a given eccentricity, N}; \]
\[ P_d = \text{axial compressive load on the section under consideration, based on 0.9·dead load + factored axial load arising from bending in coupling beams where applicable, N}; \]
\[ P_f = \text{factored axial load, N}; \]
\[ p_w = \text{vertical reinforcement ratio}; \]
\[ R = \text{VEPP} / \text{Vu}; \]
\[ \text{RCSW} = \text{Reinforced Concrete Shear Wall}; \]
\[ R_d = \text{design load resistance of reinforcement masonry member, N-mm}; \]
\[ \text{RMSW} = \text{Reinforced Masonry Shear Wall}; \]
\[ s = \text{reinforcement spacing along length, mm}; \]
\[ S_d = \text{applied load on reinforced masonry member, N-mm}; \]
\[ t = \text{wall thickness}; \]
\[ t_{ef} = \text{effective thickness of wall flange, mm}; \]
\[ T_1 = \text{natural period of the first mode of vibration, s}; \]
\[ \text{VEPP} = \text{idealized EPP curve strength}; \]
\[ V_f = \text{factored shear demand, N}; \]
\[ v_{bm} = \text{basic type dependent shear strength of masonry, MPa}; \]
\[ v_m = \text{shear strength of masonry, MPa}; \]
\[ v_n = \text{shear stress corresponding to } V_n, \text{ MPa}; \]
\[ v_p = \text{shear strength of prestressed reinforcement, MPa}; \]
\[ v_s = \text{shear strength of non-prestressed reinforcement, MPa}; \]
\[ V^* = \text{shear demand of factored loads, N}; \]
\( V_n \) = nominal shear strength, N;

\( V_{nm} \) = nominal shear strength provided by masonry, N;

\( V_{ns} \) = nominal shear strength provided by reinforcement, N;

\( V_r \) = factored shear resistance, N;

\( V_{Rd1} \) = masonry contribution to shear resistance, N;

\( V_{Rd2} \) = shear reinforcement contribution to shear resistance, N;

\( V_{sd} \) = design value of the applied shear load, N;

\( V_u \) = load at the point of maximum load;

\( V_{0.8u} \) = load at the point of 20% strength degradation from the point of maximum load;

\( V_y \) = load at first-yield of the outermost vertical reinforcement;

\( \alpha = 1.5 \) for ordinary RMSWs; 3.0 for intermediate RMSWs; 4.0 for special RMSWs;

\( \beta_1 = 0.8 \) for \( f'_m \leq 20 \) MPa and 0.8 – 0.1 for each 10 MPa of strength in excess of 20 MPa;

\( \Delta e = \Delta_{EPP} / K_e; \)

\( \Delta_{EPP} \) = the lateral displacement where the horizontal 0.8\( V_u \) line intersects the experimental load versus displacement envelope;

\( \Delta_{max} \) = maximum displacement attained during the experimental test;

\( \Delta_{uoop} \) = out-of-plane displacement during the maximum load drift cycle;

\( \Delta_{0.8uoop} \) = out-of-plane displacement during the 20% degradation drift cycle;

\( \Delta u \) = displacement at the point of maximum load;

\( \Delta_{0.8u} \) = displacement at the point of 20% strength degradation from the point of maximum load;

\( \Delta_y \) = displacement at first-yield of the outermost vertical reinforcement;

\( \varepsilon_{mu} \) = maximum usable compressive strain of masonry, mm/mm;

\( \varepsilon_{uoop} \) = average end zone vertical strain during the maximum load drift cycle;
\( \varepsilon_{0.8uoop} \) = average end zone vertical strain during the 20% degradation drift cycle;

\( \gamma_{el} \) = seismic safety factor;

\( \gamma_m \) = partial safety factor for masonry properties;

\( \gamma_s \) = partial safety factor for steel;

\( \phi \) = strength reduction factor;

\( \phi_m \) = resistance factor for masonry;

\( \phi_s \) = resistance factor for steel;

\( \sigma_d \) = the design compressive stress perpendicular to the shear in the member at the level under consideration, using the appropriate load combination, MPa;

\( \gamma_m \) = partial safety factor for masonry properties;

\( \mu \) = coefficient of friction taken as 1.0 for masonry-to-masonry or masonry-to-roughened-concrete sliding planes and 0.7 for masonry-to-smooth-concrete/masonry-to-steel sliding planes;

\( \mu_u \) = the ultimate ductility factor = 1 \( / R \);

\( \rho_v \) = average vertical reinforcement ratio over wall length;

\( \rho_h \) = average horizontal reinforcement ratio over wall height;

\( \xi_{critical} \) = critical out-of-plane displacement (normalized to wall thickness, \( t \)) prior to instability;

\( \theta_p \) = plastic hinge rotational deformation capacity, mm;

\( \chi \) = factor used to account for the direction of compressive stress in a masonry member relative to the direction used for the determination of \( f_m' \): 0.5 for compressive forces applied normal to the vertical mortar joints and the grout is not horizontally continuous in the zone of compression; 0.7 for compressive forces applied normal to the vertical mortar joints and the grout is horizontally continuous in the zone of compression; 1.0 for compressive forces applied normal to the horizontal mortar joints.
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Dedication

To my wife, Angelique, and daughter, Dacia.
Chapter 1: Introduction

1.1 Research Motivation

Reinforced masonry shear walls (RMSWs) have often been used as the seismic-force-resisting system (SFRS) for masonry buildings in regions of moderate to high seismic hazard. Several experimental studies related to the seismic response of RMSWs have been conducted over the last 40 years [10]-[16] and based on these studies, it has been preliminarily established that appropriately designed and detailed RMSWs can adequately accommodate inertial force and displacement demands due to seismic action through ductile flexure-dominant behaviour. A large proportion of common RMSW applications involve buildings that require tall, slender walls with a post-disaster level of performance such as pump stations, fire halls, and school gymnasiums. This study defines walls with these attributes as ductile slender masonry shear walls (DSRMSWs), which are designed as “Ductile Shear Walls” (the most ductile class of RMSW in Canada) in accordance with the Canadian masonry design code’s (CSA S304-14 [17]) detailing requirements (or similar ductile detailing requirements provided by other masonry design codes), but geometrically characterized as having a height-to-length \( h_w/L_w \) ratio greater than 1.5, and having a height-to-thickness \( h_w/t_w \) ratio higher than 20. The minimum \( h_w/L_w \) ratio of 1.5 was selected since DSRMSWs should generally be governed by flexure-dominant behaviour, and a \( h_w/L_w \) ratio of 1.5 or greater is generally required to achieve this. The minimum \( h_w/t_w \) ratio of 20 was selected to investigate the degree of conservatism of the current \( h_w/t_w \) limits provided by CSA S304-14 [17] since this value significantly exceeds the code prescribed values, which range from 12 to 16 for “Ductile Shear Walls.” This latter point provided one of the primary impetuses for this research study, as described later in this chapter.
Despite the relatively high demand for these walls in the Canadian masonry construction practice, their seismic response remains relatively not well understood compared to other types of RMSWs. This is primarily because the majority of recent experimental testing on RMSWs has been conducted using specimens that are either squat or non-slender (thus not classified as DSRMSWs), or do not reflect RMSWs that are typically used in current practice in terms of vertical reinforcement content, end zone confinement, types of boundary elements, or other design parameters. Moreover, there is additional uncertainty surrounding the lateral stability of DSRMWs that are subjected to seismic effects due to limited available experimental data. This uncertainty is believed to have contributed to the potentially conservative $h_u/t_w$ restrictions imposed on new DSRMSWs by CSA S304-14 [17]. Despite a lack of experimental evidence supporting the $h_u/t_w$ restrictions, these detailing requirements have had a significant impact by severely reducing the economic viability of DSRMSWs for many of their formerly common applications by forcing the use of walls with excessive thickness. For example, while concrete wall thicknesses may be varied relatively easily, RMSW thicknesses can only increase incrementally based on the masonry unit thickness or number of masonry units making up the overall wall thickness. It is therefore crucial that both the in-plane and out-of-plane performance of DSRMSWs be investigated to improve the understanding of the behaviour of these walls when subjected to in-plane seismic effects.

In general, DSRMSWs subjected to seismic effects initially demonstrate either a pure flexure failure mode or a combined shear and flexure (shear-flexure) mode. A pure flexure failure is characterized by yielding of the vertical reinforcement at the tension end zone and limited toe crushing at the compression end zone (see Figure 5a). While a combined shear and flexure mode (shear-flexure) is characterized by yielding of both the vertical and horizontal reinforcement (see Figure 5b). However, each of these initial flexure-dominant behaviour types may also precipitate or interact with secondary flexure-related failure modes, including sliding, toe-crushing, bar-buckling, rocking/bond-slip, or lateral
instability. While a diagonal tension shear failure mode is also possible for DSRMSWs, masonry design codes, including CSA S304-14 [17], specifically provide provisions to properly design and detail DSRMSWs such that the likelihood of diagonal tension shear failure is minimized. Classification of the different failure modes affecting DSRMSWs is however not concisely documented, and the interaction between these failure modes is currently not well covered in past research.

Beginning in the 1970s, some international design codes have addressed many of these failure modes in their seismic design provisions for DSRMSWs that are subjected to the combined effects of gravity and lateral loading, and these provisions have typically evolved significantly over time. However, the current DSRMSW provisions tend to vary from code to code in terms of both complexity and conservatism, and some codes do not even address all potential prominent failure modes. Comparison of leading design codes using experimental test results would also present a significant benefit to help understand what code provision methodologies provide the best agreement with the experimental data.

Figure 5. Primary flexural failure modes: (a) ductile flexure and (b) combined shear and flexure (shear-flexure).
In Canada, DSRMSWs are designed to resist seismic effects by ensuring that their lateral load capacity, as determined by the provisions of CSA S304-14 [17], exceeds the expected lateral load demands prescribed by the National Building Code of Canada (NBCC 2015 [18]). Currently, both CSA S304-14 [17] and NBCC 2015 [18] primarily use force-based approaches to determine both the capacities and demands of DSRMSWs; however, there is a current trend towards designing structural components for seismic demands by following a displacement-based performance design approach (performance-based design or PBD) instead. As an example, Section 4 – Seismic Design of the Canadian bridge design code of CSA S6-14 [19], which is currently used for the seismic design of bridges, has been continuously transitioning from initially using an equivalent static force approach to now requiring PBD for many different bridge category types. Buildings designed using a PBD approach must meet certain measurable or predictable performance requirements when subjected to the design loading. The building’s structural components must also meet performance requirements, such as limits on base rotations, strains of wall end zones, and permissible drifts, which dictate the PBD capacity of the component. Although the procedures for determining the force-based demands and capacities of DSRMSWs are well-accepted, similar methods for determining PBD demands and capacities are still being established. Current engineering practice, therefore, requires practical design tools that can reasonably estimate the displacement-based performance of DSRMSWs, of which few have the necessary combination of user-friendliness, capacity to capture shear deformations, overall robustness for typical DSRMSWs, and accuracy for use by practicing design engineers at this time.

1.2 Research Goal and Objectives

The goals of the study documented in this dissertation are to improve the state of knowledge regarding the performance of DSRMSWs under in-plane seismic effects and to provide practicing design engineers
a background and an analysis tool to help determine the PBD capacity of these walls. The main objectives to achieve these goals are as follows:

1. Conduct an experimental test series of DSRMSW specimens to observe and record the behaviour of these walls when subjected to different forms of in-plane seismic effects.

2. Analyze the in-plane performance of DSRMSWs subjected to in-plane seismic loads to investigate the typical displacement-based performance characteristics of these walls to assess the appropriateness of the ductility-related modification factor used by CSA S304-14 [17] and NBCC 2015 [18].

3. Determine the key parameters influencing the out-of-plane displacement response of DSRMSWs subjected to in-plane seismic effects and evaluate the suitability of the current $h_0/t_w$ restrictions prescribed by CSA S304-14 [17] to assess the lateral stability of DSRMSWs.

4. Classify the prominent failure modes affecting DSRMSWs and determine criteria for their occurrence as well as their effects on the wall performance and influence on other DSRMSW failure modes.

5. Compare leading current masonry design codes using the test data generated during the experimental phase of the research and summarize the code provisions that produce load capacity predictions that are in closest agreement with the experimental results.

6. Propose, calibrate, analyze, and validate a practical modelling approach that can be used to estimate the in-plane lateral load-displacement response of DSRMSWs with design parameters similar to those proposed in this study.
1.3 Thesis Outline

A multi-phase research program, including both experimental and analytical phases, was undertaken to achieve the objectives listed above. This dissertation, documenting the study, is organized as follows:

Chapter 2, Background and Literature Review: Presentation of typical characteristics of DSRMSW buildings and the background to the initiation of this study. Past research on experimental studies focused on DSRMSWs is reviewed, followed by a summary of relevant analytical nonlinear modelling approaches used to predict the lateral load-displacement response of these DSRMSWs.

Chapter 3, Experimental Program: Eight full-scale DSRMSWs wall specimens were constructed in the laboratory and subjected to a reversed-cyclic quasi-static loading protocol (please note that two of the wall specimens were constructed and tested during the author’s M.A.Sc. thesis work). Details of the specimen construction, experiment test setup, instrumentation, and the loading protocol are described.

Chapter 4, Experimental Results: The wall specimens were each tested using incrementally increasing displacement demands until failure (defined as at least a 40% reduction from the peak lateral load capacity). Detailed observations of the specimens are provided, followed by a presentation and analysis of the in-plane and out-of-plane responses.

Chapter 5, DSRMSW Failure Modes: An overview of the most prominent in-plane flexure-related failure modes affecting DSRMSWs, and their relation to one another based on the findings from the experimental phase of this study is provided. Subsequently, the seismic code provisions for DSRMSWs from several leading international masonry design codes are first summarized and then compared by analyzing their predicted nominal ULS lateral load capacities relative to the experimental values obtained from the experimental phase of the study.
Chapter 6, Analytical Model: The construction of the proposed nonlinear modelling approach is described, followed by a presentation of the parametric and sensitivity analyses at both the microscopic and macroscopic modelling levels using the results from the experimental phase of the study. The outcomes of the analyses provide a set of recommended hysteretic parameters and proposed element and spring discretization levels, which are then tested in a numerical validation to predict the response of DSRMSW specimens tested by others.

Chapter 7, Conclusions: The conclusions and significant contributions of this research study are summarized, and an outline of recommendations for future research studies is presented.
Chapter 2: Background and Literature Review

2.1 Introduction

This chapter briefly presents the typical characteristics of DSRMSW buildings and the background to the initiation of this research study. Past research on experimental studies focused on DSRMSWs is reviewed, followed by a summary of relevant analytical modelling approaches used to predict the nonlinear lateral load-displacement response of DSRMSWs.

2.2 Features of DSRMSWs

DSRMSWs offer an economic advantage over similar reinforced concrete shear walls (RCSWs) for shorter structures since they require significantly less formwork and reinforcement layout effort. However, as the building height increases, the additional costs of formwork for RCSWs becomes averaged over an enlarged wall surface area if the formwork can be reused, and hence RCSWs are typically more economical for taller structures since they can also be installed at a faster rate if the floorplans are planned in a typical repetitive manner. As a result, common applications for DSRMSWs have traditionally included pump stations, fire halls, gymnasiums, warehouses, and other single-storey structures, although there is also a comparatively small contingent of low- to mid-rise residential and commercial structures that use DSRMSWs as well. For these structures, the clear height requirements are often over 4 m to provide headroom for sporting purposes, overhead rail cranes, large equipment, machinery, and other applications that require tall open spaces.

In general, RMSWs are composed of six fundamental components: masonry units/face shells, grout, vertical and horizontal reinforcement, and header and bed mortar joints, as shown in Figure 6. In Canada, CSA A371-14 [20], CSA A165 [21], and CSA A179 [22] provide the standards for wall
construction, block fabrication, and mortar and grout composition, respectively, whereas CSA S304-14 [17] and NBCC 2015 [18] are used to determine the demands and capacities of RMSWs such that each of the prominent failure modes affecting these walls are adequately protected against. CSA S304-14 [17] and NBCC 2015 [18] classify RMSWs based on their ability to dissipate energy during earthquake shaking as “Conventional Masonry Shear Walls,” “Limited Ductility Shear Walls,” “Moderately Ductile Shear Walls,” or “Ductile Shear Walls,” each with a corresponding ductility-related force modification factor, $R_d$, of 1.5, 1.5, 2.0, and 3.0, respectively (note that increasing $R_d$ values implies improved performance and that an $R_d$ of 1.0 denotes elastic seismic response). NBCC 2015 [18] requires a minimum $R_d$ of 2.0 for all post-disaster buildings, thus mandating the use of Moderately Ductile Shear Walls or Ductile Shear Walls for these applications, many of which are traditionally common applications for DSRMSWs (e.g. fire halls and pump stations).

Figure 6. Typical DSRMSW cross-section.
One of the most important CSA S304-14 [17] RMSW seismic detailing requirements for the masonry industry is based on the wall slenderness in terms of $h_u/t_w$ ratios, which range from 12 for Ductile Shear Walls to 20 for Moderately Ductile Shear Walls. These $h_u/t_w$ limits were initially established to ensure the stability of shear walls under the effects of gravity loading in combination with in-plane shears and overturning moments from seismic inertial forces, as shown in Figure 7. These $h_u/t_w$ limits are based on an analytical approach initially proposed by Paulay and Priestley [23] for estimating the minimum wall thickness required to avoid global lateral instability for a wall of a given unsupported height between lateral supports – typically existing in the form of floor and roof diaphragms. Their proposed methodology prescribes a minimum required thickness that depends on several parameters, including the longitudinal reinforcement ratio, curvature and displacement ductility demands, plastic hinge height, and the mechanical properties of the reinforcement and masonry materials [23].

![Diagram of DSRMSW subjected to in-plane seismic loading]

Figure 7. Lateral instability in a DSRMSW subjected to in-plane seismic loading.

**Equations:**

$T = \text{Flexural tension}$

$C = \text{Flexural compression}$

$M = \text{Overturning moment}$

$V = \text{Shear}$

$P = \text{Uniformly distributed axial compression}$

*Shaded areas indicate the wall end zone regions, which are susceptible to out-of-plane instability*
These $h_w/t_w$ ratios restrictions have a significant effect on the use of DSRMSWs for many of the traditionally common DSRMSW applications mentioned earlier; often resulting in DSRMSWs no longer being cost-competitive with other SFRSs. Despite the impact of these restrictions, there is only a limited body of experimental evidence related to lateral instability failure modes that occur in DSRMSWs prior to these walls attaining their ultimate lateral load capacity. Moreover, there has been no recorded evidence of this type of lateral instability in the aftermath of actual earthquakes. This relative lack of experimental data provided the initial impetus for the study presented in this dissertation and therefore carried significant importance in the early stages of its development. The following sections provide a literature review of experimental and analytical studies on DSRMSWs and RCSWs that experienced at least one class of lateral instability, as originally defined by Vallenas et al. in 1979 [26] during their research on RCSWs (see Figure 8):

- **Class A**: localized instability due to buckling of the vertical reinforcement within the wall end zone following spalling of the concrete cover (in RCSWs) or face shells (in DSRMSWs). The global wall stability is usually preserved after a Class A instability takes place.

- **Class B**: global buckling of a wall end zone following spalling of the concrete cover (in RCSWs) or face shells (in DSRMSWs); this mechanism results in the global collapse of the wall.

- **Class C**: global buckling of a wall end zone prior to spalling of the concrete cover (in RCSWs) or the face shells (in DSRMSWs); this mechanism is characterized by open residual cracks over the plastic hinge height and culminates in global collapse of the wall.
2.3 Experimental Studies

The available experimental evidence related to lateral instability in DSRMSWs is somewhat limited, as documented by Azimikor et al. (2012 [24]) and Herrick (2014 [25]). In most cases, out-of-plane deflections and lateral instability only occurred in the test specimens at high lateral drift and ductility levels that are beyond the practical relevance for the performance of DSRMSWs designed to current masonry design code requirements, which are typically forced-based. Relevant experimental research studies are summarized in this section to document the occurrence of various lateral instability mechanisms in not just DSRMSWs, but also RCSWs that have only a single layer of vertical reinforcement, which are similar to DSRMSWs and help expand the experimental conclusions developed to date on the subject. The summary shows that Class A
instability was the most commonly observed lateral instability mechanism in past experimental studies, whereas there only a few studies with documented evidence of Class B instability in DSRMSWs or RCSWs with a single layer of vertical reinforcement. Interestingly, the occurrence of Class C instability in reinforced masonry (RM) or reinforced concrete (RC) specimens with a single layer of vertical reinforcement have been limited to only studies on prismatic specimens subjected to uniaxial cyclic tension/compression loading.

The reviewed studies are classified into i) tests on RCSW/DSRMSWs subjected to reversed-cyclic in-plane loading, and ii) tests on RC or RM uniaxially-loaded prismatic specimens. It should be noted that the studies on uniaxially-loaded prismatic specimens have been considered in this review to aid with the understanding of Class C instability and help identify key factors that influence its development. It is acknowledged that uniaxially-loaded prismatic specimens are not capable of simulating the actual boundary conditions over the height of the wall end zone; thus the test results do not take into account the effect of strain gradient along the wall length or height.

2.3.1 Tests on DSRMSWs and RCSWs Subjected to Reversed-Cyclic Loading

Shedid et al. [27], [28] observed Class B instability in two tests on full-scale rectangular DSRMSW specimens subjected to reversed-cyclic loading. These specimens were part of a 13-specimen test series that was used to evaluate the effect of reinforcement ratio and applied axial compressive stress on DSRMSW behaviour. A fully reversed-cyclic lateral loading protocol and an axial post-tensioning load were applied at the top of each specimen. The two specimens that experienced Class B lateral instability (named Wall 4 and W4, from two separate test series) had $h_u/t_w$ ratios of 13.7 and 18.9, $h_u/L_w$ ratios of 1.5 and 2, vertical reinforcement ratios of 1.17 and 1.31%, and were loaded with low levels of applied axial stress ($0.06f'_m$ and zero), respectively. The Class B
instabilities in each specimen occurred at displacement ductility levels of approximately 3.5 and 10, respectively. At that stage, the specimens experienced significant toe-crushing damage and showed strength degradation levels of roughly 30% and 40% relative to the ultimate capacity, respectively. In both specimens the failure was attributed to buckling of large-sized vertical reinforcing bars (25 mm diameter), which pulled an intact portion of the wall surrounding the crushed regions out of plane, leading to termination of the tests.

He and Priestley [29] observed Class B instability during a test on a full-scale T-shaped DSRMSW specimen subjected to reversed-cyclic loading. In total, nine specimens were tested using quasi-static reversed-cyclic and dynamic loading protocols with axial post-tensioning loads applied at the top of each specimen. Specimen F4, one of the quasi-static specimens, experienced Class B lateral instability and had a $h_d/t_w$ ratio of 25.6, a $h_d/L_w$ ratio of 3.1, a vertical reinforcement ratio of 0.55%, and was loaded with applied axial stress of $0.09f'_m$. The specimen was retrofitted with confining plates in the horizontal mortar joints over the lower courses of the web, which were found to significantly increase the displacement ductility of the wall at ultimate lateral load capacity. The Class B instability failure only occurred after face shell splitting and spalling took place, which exposed the vertical reinforcing bars in the bottom course of the web end zone during the penultimate load cycle. During the final load cycle, the bottom portion of the wall rapidly translated out-of-plane to such a degree that all lateral load-carrying capacity of the wall was lost. The displacement ductility at failure was approximately 8.0 in the direction, causing tension in the web, and the corresponding strength was at about 70% of the ultimate capacity.

Almeida et al. [30] observed significant out-of-plane behaviour in an RCSW T-shaped two-third scale specimen TW1 with a single-layer of vertical reinforcement which was subjected to reversed cyclic loading. This specimen was part of a 5-specimen test series that assessed the
influence of wall thickness on member stability, the role of lap splices on damage distribution and displacement ductility, and the effects of simultaneous application of out-of-plane loading on member response. The scaled specimens represented the bottom story of a multi-storey prototype wall, and the impact of upper storey forces was simulated through in-plane loading applied through a series of actuators. Specimen TW1 had a \( h_u/t_w \) ratio of 25, a \( h_u/L_w \) of 3.7, a total vertical reinforcement ratio of 0.67% (note that the vertical reinforcement ratio was 2.63% in the boundary elements) and was subjected to an applied axial precompression of 0.04\( f'_c \). The specimen experienced significant out-of-plane displacements of nearly 60% of the wall thickness during the penultimate load cycle. These displacements were almost fully recovered at the time of failure [31]. The wall had reached a displacement ductility of approximately 7.0 at failure, but it experienced only minor strength degradation relative to the maximum load. The failure mode was characterized by toe-crushing and Class A instability bar buckling, although the specimen behaviour was significantly influenced by the out-of-plane displacements that occurred before the final failure.

### 2.3.2 Tests on RC and RM Uniaxially-Loaded Prismatic Specimens under Uniaxial Tension/Compression Cyclic Loading

Class C instability was observed by Rosso et al. [32], [33] during the reversed-cyclic testing of three RC prismatic specimens with a single layer of reinforcement. The specimens were geometrically identical, had a \( h_u/t_w \) ratio of 24, a \( h_u/L_w \) ratio of 3, and a vertical reinforcement ratio of 2.01%. The specimens experienced large out-of-plane displacements beyond one-half of the wall thickness. The crushing occurred in the specimens after excessive out-of-plane displacements caused instability following an axial tensile strain demand of 0.0075.
The only experimental evidence of a Class C instability in RM specimens is limited to three full-scale prismatic specimens tested by Azimikor et al. [34], [35]. The specimens represented the end zone of a DSRMSW and were part of a 5-specimen test series examining the effects of vertical reinforcement ratio and plastic hinge height on the out-of-plane response of wall end zones. The specimens were subjected to an asymmetric reversed-cyclic uniaxial loading protocol, with incrementally increasing uniaxial tension excursions, and uniaxial compression excursions that were kept constant at much smaller imposed strains than the tension excursions. Specimens C2, C3, and C4, which experienced Class C instability had a $h_u/t_w$ ratio of 27.1, a $h_u/L_w$ of 6.3, and vertical reinforcement ratios of 0.48%, 0.71% and 1.07%, respectively. Some of the specimens were retrofitted with glass fibre reinforced polymer wrapping at varying lengths from their top ends, which resulted in the effective plastic hinge heights being limited to the middle portion of the specimens. Since the specimens were not subjected to axial precompression; thus, the testing more closely represented the flexural compression at an end zone of a lightly axially-loaded DSRMSW. It was observed that after uniform cracks had formed over the specimen height during the tension excursion, out-of-plane rotations formed soon after load reversal into a compression excursion; this produced large out-of-plane displacements that eventually culminated in lateral instability. The average axial tensile strains attained before the onset of instability were 0.0079, 0.0080, and 0.0169 for specimens C2, C3, and C4, respectively.

2.4 Analytical Modelling Approaches for DSRMSWs

While performance-based design (PBD) demands on DSRMSWs are highly dependent on geographical location, the PBD capacities are mostly independent of this factor and practicing design engineers may select from several nonlinear modelling approaches available to produce
PBD capacity estimations. Hand calculations based on strut-and-tie models or beam theory with plastic hinge height assumptions are available from design codes, books, and manuals for relatively simple applications (e.g. [12], [36]). Hand calculation-based approaches could be considered as a form of macroscopic modelling and carry the advantage of being fast and easy to understand conceptually; however, their accuracy is relatively low, most do not account for shear deformations, and they are not always robust for complicated geometries. The next-simplest PBD capacity estimation approach uses numerical truss element models that discretize a wall element into a series of cells with diagonal truss elements that represent the diagonal compression field (e.g. [37], [26]). This approach is typically a macroscopic approach and is simple to understand but may be relatively cumbersome to implement due to the often large number of elements involved. Beam-column models are generally the next-simplest numerical capacity prediction approach, and are another form of macroscopic modelling that has been implemented since the 1960s and the 1970s and can take the form of lumped-plasticity models that focus inelasticity at plastic hinge locations (e.g. [26], [38]-[40]) or distributed plasticity models that integrate nonlinear effects over the length of the elements (e.g. [41], [42]). Beam-column models are simple, relatively robust, and involve a relatively low degree of modelling effort; however, they are best suited to tall walls with high $h_o/L_w$ ratios where the lateral deformations are dominated by flexure since shear deformations are not captured with a typical beam-column model. In general, unlike all previously-mentioned PBD capacity estimation methodologies, numerical shell element models are directly capable of capturing shear behaviour and typically use a smeared or discrete cracking approach for modelling the masonry base material, and either smeared or discrete reinforcement within the masonry (e.g. [43]-[45]). Shell element models can provide a high degree of accuracy, though at a relatively high computational cost and would often be considered as a form of
microscopic modelling. Numerical solid elements are used as another microscopic modelling approach that provides an additional level of complexity that can include explicitly three-dimensional elements with surrounding spring and contact elements that represent the mortar connections between the individual masonry units (e.g. [46], [47]). With careful model development, verification, and validation, solid elements can be used to produce a high degree of accuracy and precision for some applications, although typically at very high levels of computational time and with a lack of robustness.

One other numerical macroscopic modelling approach to produce PBD capacity estimations is the use of MVLEs. This type of model is simple to understand conceptually, carries a relatively low level of modelling effort, is capable of capturing shear deformations, is robust for typical DSRMSW applications, and has good accuracy for modest computational time. Kabeyasawa originally developed the MVLE model (MVLEM) approach at al. in 1983 [48] and subsequently expanded by Vulcano et al. [49], Fischinger et al. [50], Colotti [51], Linde and Bachmann [52], among others. The MVLEM has been used to capture the lateral load-deformation response of RCSWs successfully (e.g. [53]-[56]). However, this modelling approach has not yet been implemented for DSRMSWs. Similar to a fibre element often used in beam-column models, the MVLEM consists of a series of uniaxial elements, or microfibers, connected to infinitely rigid beams at the top and bottom of the element, as well as a horizontal spring placed at the centre of rotation of the element. In this model, uniaxial elements simulate the flexural response, while the horizontal spring simulates the shear response (i.e. the flexural and shear responses are uncoupled). A structural wall is typically modelled as a stack of MVLEs, which are placed one upon the other, as shown in Figure 9. The MVLEM is capable of capturing shifting of the neutral axis during reversed-cyclic loading as well as the effect of fluctuating axial force on the strength and stiffness.
It is also possible to incorporate various steel and concrete hysteretic models for the uniaxial spring elements and thus may easily account for local effects such as confinement of the longitudinal reinforcement or potentially the effect of out-of-plane displacements on the vertical stiffness of an end zone.

Figure 9. A tall DSRMSW modelled using MVLEs.

2.5 Summary and Conclusions

Based on the literature review of past DSRMSW experimental studies, there is a definite lack of full-scale experimental tests on DSRMSWs that experience global lateral instability classes, and
currently, only uniaxially-loaded prismatic specimens have demonstrated Class C instability. The lack of experimental data for this critical aspect of DSRMSWs, when considering the limitations on these walls due to code $h_0/t_w$ restrictions, provided an impetus for the experimental program discussed in the Chapters 3 and 4, and the failure mode and code comparison study presented in Chapter 5.

Similarly, while there are a number of modelling techniques available for practicing DSRMSW design engineers wishing to follow a PBD design approach, there is a void to fill for a modelling method that is simple to implement, capable of capturing all significant deformation types, robust for typical DSRMSW applications, while still achieving good accuracy for the critical aspects of the lateral load-displacement response. The nonlinear MVLEM proposed in Chapter 6 helps fill that void.
Chapter 3: Experimental Program

3.1 Introduction

Full-scale DSRMSWs wall specimens were constructed in the laboratory and subjected to a reversed-cyclic quasi-static loading protocol (please note that two of the wall specimens were built and tested during the author’s M.A.Sc. thesis work). Details of the specimen construction, experiment test setup, instrumentation, and the loading protocol are described in this chapter.

3.2 Test Specimens

The wall specimens were all DSRMSWs designed with good agreement with the current practice in the Canadian masonry construction industry. Most of the specimens were designed as tall, single-storey walls that represent typical masonry buildings such as warehouses and pump stations, as previously discussed. However, specimen W1 was designed as the bottom storey of a multi-storey low-rise residential masonry building and hence was subjected to a modest level of sustained axial compression as well as shear and overturning moments that were representative of a taller wall that could not fit within the space constraints of the UBC Structures Laboratory (see Figure 6). This simulation was achieved by slaving the loads in the load-controlled vertical actuators to the load in the displacement-controlled horizontal actuator, all of which were connected to a single load beam. As the horizontal actuator pushed in one direction, one vertical actuator would increase its applied load and the other vertical actuator would decrease its applied load to increase the applied shear and moment demands while maintaining a constant axial load. Note that the effect of axial compression reduces the likelihood of a lateral instability failure mode.
The experimental phase of the research was initially planned to induce lateral instability failure mechanisms in a series of eight full-scale wall specimens. Six of the specimens were rectangular-shaped with varying $h_u/L_w$ and $h_u/t_w$ ratios, amount and distribution of reinforcement, and axial stress levels. The remaining two specimens were T-shaped walls subjected to different loading protocol types (symmetrical and asymmetrical cyclic loading). Table 1 presents the specimen test matrix, which includes the mean masonry compressive strength ($f'_m$), axial stress ratio ($P/A_vf'_m$), vertical and horizontal reinforcement ratios ($\rho_v$ and $\rho_h$), and shear stress ratio ($V_{max}/A_nf'_m^{0.5}$). Note that the 4000 lb tributary dead load from the horizontal actuator is not included in the axial precompression column. Plan and elevation views of the test specimens are presented in Figure 11.

![Simulated Wall](image1)

![Wall Specimen](image2)

**Figure 10.** Specimen W1: the simulated wall from a hypothetical low-rise residential masonry building and the wall specimen tested in the laboratory.
The first design objective was to achieve flexure-dominated behaviour, which is characterized by high tensile strains in the vertical reinforcement over a distinct region of plastic hinging; this is a theoretical prerequisite for the development of lateral instability. Given the fact that the height of the plastic hinge strongly depends on the wall-length [12], the wall $h_u/L_w$ ratios of the specimens were maintained in the range from 1.5 to 3.0, which permitted relatively large plastic hinge heights relative to the specimen height, while still promoting a flexural response. In order to evaluate the CSA S304-14 [17] $h_u/t_w$ limits, relatively high $h_u/t_w$ ratios in the range of 21 to 29 were selected. The specimen height was kept approximately constant at 3.8 to 4 m (due to the height constraints of the testing facility); thus, the $h_u/t_w$ ratios were mainly a function of the wall thickness. The masonry units consisted of standard Canadian concrete blocks (400 mm length x 200 mm height), with two hollow cores and a nominal thickness of either 150 or 200 mm (140 or 190 mm in terms of actual thickness, respectively), depending on the test specimen. The masonry was laid in 50% running bond using Type S mortar for face shell bedding.

Table 1. Specimen Test Matrix.

<table>
<thead>
<tr>
<th>ID</th>
<th>Cross-Section</th>
<th>$h_u$ (mm)</th>
<th>$h_u/t_w$</th>
<th>$f_m$ (MPa)</th>
<th>$p$</th>
<th>$\rho_v$ (%)</th>
<th>$\rho_h$ (%)</th>
<th>$V_{max}$</th>
<th>$A_{nv}f_m$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1</td>
<td>R</td>
<td>3800</td>
<td>1.5</td>
<td>23.4</td>
<td>0.08</td>
<td>0.33</td>
<td>0.36</td>
<td>0.15</td>
<td></td>
</tr>
<tr>
<td>W2</td>
<td>R</td>
<td>3800</td>
<td>1.5</td>
<td>23.4</td>
<td>0</td>
<td>0.33</td>
<td>0.36</td>
<td>0.13</td>
<td></td>
</tr>
<tr>
<td>W3</td>
<td>R</td>
<td>4000</td>
<td>21.1</td>
<td>27.1</td>
<td>0</td>
<td>0.24</td>
<td>0.26</td>
<td>0.08</td>
<td></td>
</tr>
<tr>
<td>W4</td>
<td>R</td>
<td>4000</td>
<td>21.1</td>
<td>27.1</td>
<td>0</td>
<td>0.15</td>
<td>0.26</td>
<td>0.03</td>
<td></td>
</tr>
<tr>
<td>W5</td>
<td>R</td>
<td>4000</td>
<td>28.6</td>
<td>27.1</td>
<td>0</td>
<td>0.33</td>
<td>0.36</td>
<td>0.10</td>
<td></td>
</tr>
<tr>
<td>W6</td>
<td>T</td>
<td>4000</td>
<td>21.1</td>
<td>26.4</td>
<td>0</td>
<td>0.27</td>
<td>0.26</td>
<td>0.09</td>
<td></td>
</tr>
<tr>
<td>W7</td>
<td>T</td>
<td>4000</td>
<td>21.1</td>
<td>26.4</td>
<td>0</td>
<td>0.27</td>
<td>0.26</td>
<td>0.14</td>
<td></td>
</tr>
<tr>
<td>W8</td>
<td>R</td>
<td>4000</td>
<td>21.1</td>
<td>26.4</td>
<td>0</td>
<td>0.24</td>
<td>0.26</td>
<td>0.09</td>
<td></td>
</tr>
</tbody>
</table>

Notes

$A_{nv}$ Net cross-sectional area subjected to shear forces
$A_n$ Net cross-sectional area subjected to axial forces

R Rectangular-shaped
T Tee-shaped

ID

21.1
21.1
21.1
Cross-Section

27.1
27.1
21.1
21.1
28.6
Figure 11. Plan and elevation views of the wall specimen W1, W2, and W5 [units in mm].
Figure 12. Plan and elevation views of the wall specimen W3 [units in mm].
Figure 13. Plan and elevation views of the wall specimen W4 [units in mm].
Figure 14. Plan and elevation views of the wall specimens W6 and W7 [units in mm].
Figure 15. Plan and elevation views of the wall specimen W8 [units in mm].
To induce a Class C instability in specimens W6 and W7, a stress reduction at the extreme compression fibre was accomplished by increasing the effective compression zone width through flanged boundary elements, as shown in Figure 16. The strain distribution in specimen W6 was further manipulated using an asymmetric loading protocol that incrementally increased displacement/drift amplitudes for loading cycles toward the flanged end zone but held constant the amplitudes for loading toward the web end zone, as discussed in more detail in the following sections. This enabled the development of high tensile strains (theoretically up to 4%) at the web end zone while maintaining low compressive strains during the load reversals.

![Figure 16. Specimens W6 and W7 with flanged boundary elements: a) wall elevation and b) plan [units in mm].](image)

Custom-designed sliding restraints acting as passive bumper-like restraints were provided at the wall toes of specimens W7 and W8 to reduce potential sliding displacement during the testing. The motivation for introducing these sliding restraints was the occurrence of sliding at the base of most of the specimens tested previously, although this may not be representative of most actual walls. Significant sliding displacements, especially in the specimens without axial
precompression, caused in-plane shear and rotational deformation of the vertical reinforcement, which the author’s believed help preclude the development of the out-of-plane bar rotations necessary to produce significant out-of-plane displacements of the wall end zone. Figure 17 shows the restraints, which were in the form of stiffened HSS 152x152x13 sections for the web end zone and HSS 203x102x13 for the flange end zone. These steel sections were fastened to the bearing plates of the outermost vertical post-tensioning rods connecting the footings to the reaction floor. The restraints were intended to be in contact only with the end faces of the specimens and were purposely not post-tensioned in order to prevent significant confinement which would affect the flexural performance of the wall toes.

![Image of sliding restraints](image)

**Figure 17. Sliding restraints: a) at a flanged end zone, and b) at a web end zone.**

The specimens were reinforced with horizontal and vertical reinforcement adhering to the CSA S304-14 [17] ductile detailing requirements for Ductile Shear Walls (characterized by the ductility-related force modification factor $R_d$ of 3.0 and overstrength-related force modification
factor $R_o$ of 1.5). The amount of horizontal reinforcement was selected to prevent the occurrence of pure shear failure in the specimens, and the horizontal reinforcement ratio varied from 0.26 to 0.36%; which are values that are somewhat higher than standard Canadian masonry construction practice, however not overly so. The horizontal reinforcement consisted of 10M bars placed in bond beam blocks and terminated with 180° hooks at the ends. Additional L-shaped horizontal reinforcing bars were provided at the vertical web-flange interface of specimens W6 and W7 to prevent an interface shear failure at that location. The vertical reinforcement ratio for the wall end zones ranged from 0.15 to 0.33%; which are typical of standard Canadian masonry construction practice. The vertical reinforcement generally consisted of 15M bars concentrated in the wall end zones, and flanges where applicable, and 10M bars for the distributed reinforcement along the middle of the wall webs. The vertical bars were anchored into reinforced concrete footings and were continuous over the full wall height of the specimens without the use of lap splices. The specimens were fully grouted with a commercially available coarse grout mix. The grout was poured in two half-height lifts for specimens W1 and W2, a single lift for specimens W3 to W5 for constructability purposes, and then back to two half-height lifts for specimens W6 to W8. “Clean-out” ports were also placed along the bottom course of specimens W6 to W8 to ensure proper grout consolidation.

Material testing was performed in accordance with the appropriate Canadian Standards Association standards. Masonry unit testing was performed per the CSA A165-04 standard [21], and determined an average net compressive strength of 26.9 MPa (coefficient of variation, COV = 9.2%) for specimens was obtained based on the testing of at least five samples per wall specimen. Mortar and grout testing was performed per CSA A179-14 [22] and determined an average compressive strength of 12.8 MPa (COV = 10.0%) based on the testing of at least nine standard
50 mm mortar cubes per wall specimen. Grout samples were in the form of 100 mm-diameter x 200 mm-high cylinders cast in non-absorptive molds. The average grout compressive strength was 39.1 MPa (COV = 9.3%), based on ten samples per wall specimen. The masonry compressive strength, \( f'_m \), was determined per CSA S304-14 [17] by performing compressive strength tests on two-unit-high grouted masonry prisms. An average masonry compressive strength of 26.2 MPa (COV = 8.2%) was obtained from five two-unit-high prism tests per wall specimen. Horizontal and vertical reinforcing bars were Grade 400 steel with a nominal yield strength of 400 MPa. A minimum of 10 tensile bar tests were conducted for each wall specimen using 10M and 15M reinforcing bar coupons. The tests were performed in compliance with CSA G301.18-M92-R2002 [57]. An average uniaxial yield strength of 498.3 MPa (COV = 3.0%) was obtained from the tests, corresponding to an average yield strain of approximately 0.003. These values, while considerably higher than the 400 MPa nominal yield strength, are not uncommon in Canadian construction practice. Table 2 summarizes the average material strengths for the specimens based on the sampling described above.
Table 2. Specimen Materials Testing Summary: Average Material Strengths.

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>$f_{mu}$</th>
<th>$f_{mo}$</th>
<th>$f_{gr}$</th>
<th>$f_{m}$</th>
<th>$f_y$</th>
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<tr>
<td>W1</td>
<td>27.9</td>
<td>12.5</td>
<td>41.8</td>
<td>24.1</td>
<td>483.6</td>
</tr>
<tr>
<td>W2</td>
<td>28.8</td>
<td>11.8</td>
<td>44.1</td>
<td>23.7</td>
<td>511.5</td>
</tr>
<tr>
<td>W3</td>
<td>25.7</td>
<td>11.2</td>
<td>37.5</td>
<td>26.3</td>
<td>508.3</td>
</tr>
<tr>
<td>W4</td>
<td>22.3</td>
<td>15.7</td>
<td>32.2</td>
<td>23.9</td>
<td>465.3</td>
</tr>
<tr>
<td>W5</td>
<td>23.8</td>
<td>13.4</td>
<td>35.6</td>
<td>25.5</td>
<td>506.4</td>
</tr>
<tr>
<td>W6</td>
<td>28.1</td>
<td>12.6</td>
<td>38.8</td>
<td>28.4</td>
<td>505.2</td>
</tr>
<tr>
<td>W7</td>
<td>28.5</td>
<td>12.1</td>
<td>40.2</td>
<td>28.9</td>
<td>507.1</td>
</tr>
<tr>
<td>W8</td>
<td>29.7</td>
<td>13.4</td>
<td>42.3</td>
<td>29.1</td>
<td>498.8</td>
</tr>
</tbody>
</table>

**MEAN** | 26.9 | 12.8 | 39.1 | 26.2 | 498.3 |

**STD**  | 2.5  | 1.3  | 3.6  | 2.1  | 14.9  |

**COV**  | 9.2% | 10.0%| 9.3% | 8.2% | 3.0%  |

### 3.3 Testing Setup

A custom-designed test setup, shown in Figure 18 and Figure 19, was developed for testing the specimens and used an arrangement of either one or three actuators. The principal horizontal actuator had a load capacity of ±1000 kN, a ±343 mm stroke, and was pin-connected to a rigid reaction wall and the loading beam. Additionally, two vertically inclined actuators with ±645 kN capacity and a ±305 mm stroke were utilized only for the testing of specimen W1. As discussed in the previous section, the objective of the three-actuator test setup was to simulate the effects of axial precompression and overturning moments from the upper stories of a three-storey (9 m-tall) structure on the lower portion of a DSRMSW during seismic loading. The single-actuator test setup was designed to only represent a part of the SFRS of a single-storey structure. Only specimen W1 utilized the three-actuator setup as it was the single specimen investigating the effects of axial precompression. After it was determined that the axial precompression appeared to preclude the...
occurrence of Class B and C lateral instability mechanisms, it was decided to use the single horizontal actuator test setup for the remaining tests of specimens W2 to W8.

The load-transfer apparatus consisted of a double-channel steel loading beam for specimen W1 and a set of steel load-application plates that were secured at the top corners of specimens W2-W8 using ten horizontally-orientated high-strength post-tensioning rods. Custom-designed lateral supports were employed to prevent out-of-plane displacements at the top of the wall specimens during the testing. These supports effectively created a pinned boundary condition in the transverse direction and a roller-type restraint in the longitudinal direction. Two different support arrangements were utilized during the testing: the original support arrangement, used for specimens W1 and W2, and shown in Figure 18a and Figure 20a, was replaced by the second, used for specimens W3 to W8, and shown in Figure 18b, Figure 19b, and Figure 20b, which was capable of accommodating larger in-plane displacements than the original.
Figure 18. Isometric view of the test setup for specimens (a) W1 and (b) W2 to W5.

Figure 19. Isometric view of the test setup for specimens (a) W6 and W7, and (b) W8.
3.4 Loading Protocol

Figure 21 shows the displacement-controlled lateral loading protocol for specimens W1 to W5, W7, and W8 (although small displacement amplitude variations occurred at the later staging of some tests) and that of specimen W6 is shown in Figure 22. The loading protocols were displacement-controlled based on the recorded data from string potentiometer instruments situated at the same height as the primary horizontal actuator, as shown in Figure 23 and Figure 24. The load protocols were generally executed until the lateral load-carrying capacity of the specimen was reduced to approximately 60% of its peak capacity. For the asymmetric loading protocol of specimen W6, the displacement-controlled loading cycles, hereafter referred to as “drift cycles,” were set at increasing displacement amplitude increments ranging from 0.5 to 2.0 times $\Delta y$ during loading toward the flanged wall end zone. These increments remained constant during loading toward the web end zone after the $2.0\Delta y$ (0.37%) drift was reached. The approximate drift level at
the onset of first-yielding of the outermost vertical reinforcing bars at the web end zone of specimen W6, labelled as $\Delta_y$, was 0.18%; this is equivalent to approximately 7 mm of relative lateral displacement between the resultant actuator force and the base of the wall. This drift was initially determined by analysis and corresponded relatively well to the measured strain from the strain gauge data for specimen W1. This displacement amplitude increment was then utilized for all subsequent tests to maintain consistency. The first drift cycle was also used to check the actuator communication while the specimens were still in the elastic response range.

It was originally intended that following this initialization drift cycle, two drift cycles would be applied for each subsequent displacement increment. However, after the second 0.37% drift cycle it was determined that the additional unnecessary damage caused by this type of double-cycling might be detrimental to achieving a Class C instability and thus only a single drift cycle was applied for all the remaining displacement amplitude increments.

Specimens W1 to W5, W7, and W8 utilized a symmetric loading protocol. Displacement amplitude increments were identical to the positive portion of the W6 loading protocol, aside from the last three cycles. These final drift cycles were smaller in magnitude because the out-of-plane supports were shifted for the W1 to W5, W7, and W8 tests to permit large drift cycles in both directions (instead of only one direction). The same $\Delta_y$ drift value of 0.18% was utilized for specimens W1 to W5, W7, and W8 to maintain consistency among the tests (although the actual value from the hysteresis curves was different).

As discussed and illustrated in the previous section, all loads were applied via connections between the actuators and the loading beam secured to the top of each specimen. For specimen W1 only, the lateral loading protocol was preceded by an initial precompression load of 660 kN (corresponding to axial stress of $0.08f'_m$) that was applied by the net sum of the vertical actuator
loads, which were both load-controlled. The loads in these actuators were slaved to the load in the primary horizontal actuator, which was displacement-controlled. The bending moment created by the vertical actuators simulated the bending moment that would be induced by the net shear force from all three actuators being applied at the top of the 9 m-tall simulated wall. The goal was to execute the lateral loading protocol and at the same time to capture the effect of varying bending moment from the simulated upper stories, while also maintaining a constant net vertical component corresponding to the simulated gravity load.

Figure 21. Displacement-controlled loading protocol for the tests on specimens W1 to W8.
3.5 Wall Instrumentation

For specimens W1 to W5, an array of linear and string potentiometers was arranged such that in-plane and out-of-plane deformations could be monitored during each test (see Figure 23). The instruments were manufactured by a number of brands, including Duncan, Celesco, and Novotechnik. The instruments were calibrated by scaling the instrument readings using a variety of known displacements to develop a calibration value that was valid over a wide range of displacements. The sampling rate was generally one reading record every five seconds. Lateral in-plane displacements were recorded at the quarter-height points over the wall height at the end of the wall nearest the horizontal actuator. Out-of-plane displacement recording points were located at both wall end zones at two different spacings over the wall height: 200 mm spacing over the bottom third of the wall height and 800 mm spacing for the remainder of the wall height. One set of diagonally oriented potentiometers recorded shear deformations over the bottom third of the wall height, while a second set recorded shear deformations above that point.
Figure 23. Test setup instrumentation layout for specimens W1 to W5 [units in mm].

For specimens W6 to W8, linear and string potentiometers were arranged such that in-plane and out-of-plane translational displacements were captured and monitored during the tests (see Figure 24). In-plane lateral displacements were recorded at 400 mm vertical intervals over the wall height, as well as at the actuator location (200 mm below the top of the wall). Out-of-plane displacements were recorded at a series of vertically-oriented points located within the web end zone and at the wall mid-length for specimens W6 and W7, and only at both wall end zones for specimen W8. The vertical spacing of these points was 200 mm in the bottom one-third of the wall height and 800 mm for the upper portion of the wall. A set of diagonally-oriented potentiometers were used to record shear deformations over the full wall height, whereas a horizontally-oriented linear potentiometer at mid-length of the wall base recorded sliding displacements. Strain gauges
recorded the tensile strains in the outermost 15M vertical reinforcing bars at the second, third, and fourth mortar joints. Linear potentiometers were installed to monitor crack widths at the flange base and at the bottom nine horizontal mortar joints at the web end zone for specimens W6 and W7, and at both end zones for specimen W8. These locations were expected to experience extensive cracking during the testing.

Figure 24. Test setup instrumentation layout for specimens W6 to W8 [units in mm].
Chapter 4: Experimental Results

4.1 Introduction

The wall specimens were each tested using incrementally increasing displacement demands until failure (defined as at least a 40% reduction from the peak lateral load capacity). Detailed visual observations are provided in this chapter, followed by a presentation of the numerical results.

4.2 Behaviour of Test Specimens

All of the wall specimens demonstrated flexure-dominated behaviour as planned by design. The specimens were subjected to high drift demands, often beyond the practical level from the perspective of most design codes, in order to develop high tensile strains in the end zones which are prerequisite for global Class C instability. This section summarizes the visual observations regarding the testing of all wall specimens (specimens W1 to W8), while Figure 25, Figure 28, Figure 31, Figure 34, Figure 37, Figure 40, Figure 43, and Figure 46 provide visual summaries of the sequence of key events observed during the testing. Additional photos of the testing are provided in Appendix A at the quarter points during the testing in terms of drift cycles, i.e. at the drift cycle approximately corresponding to one-quarter of the maximum drift during the test, at one-half of the maximum drift, at three-quarters of the maximum drift, and the maximum drift.
4.2.1 Specimen W1

![Sequence of key events during the test of specimen W1.](image)

Figure 25. Sequence of key events during the test of specimen W1.

The test began with the application of the axial precompression load, followed by the lateral displacement-controlled loading protocol. Diagonal shear cracks first became visible during the 0.09% drift cycle and continued to increase in width during successive cycles. Large flexural cracks, exceeding 1 mm in width, began to dominate the deformations during the 0.55% drift cycle and were accompanied by the onset of face shell spalling at the wall toes (point a in Figure 25). The spalling became significant over the bottom two courses at both wall toes during the 0.74% drift cycle, causing a 100 mm-long portion of the outermost vertical reinforcing bar at the negative-direction toe to become exposed (point b in Figure 25). At that stage, out-of-plane displacements remained nearly negligible, with a maximum value of approximately 3 mm. The face shell spalling and toe-crushing damage caused the compression zone of the wall to shift toward its mid-length.
After the 1.11% drift cycle in the positive-direction, load reversal brought about a sudden compression crushing failure over the bottom two courses along most of the wall length (point c in Figure 25). The failure occurred due to high localized axial compression stresses acting over the small central portion of the wall that had not yet been affected by toe-crushing. Although the wall abruptly shifted out-of-plane during the failure, this was not considered a lateral instability failure mechanism. Figure 26 shows the lateral load-displacement of specimen W1, while Figure 27 summarizes the crack patterns recorded during the test.

Figure 26. Lateral load-displacement hysteresis for specimen W1.
Figure 27. Crack diagrams of the cracking patterns recorded during the test of specimen W1.
Small flexural cracks concentrated over the bottom six courses of the wall height developed almost immediately during the first load cycle at 0.09% drift. Further loading caused widening of these cracks and the development of many new diagonal shear cracks over approximately the bottom two-thirds of the wall height (point a in Figure 28). By the end of the 0.37% drift cycle, the cracks developed along nearly all mortar joints over the bottom 80% of the wall height. Most cracks were small at this stage, with the widest crack being 1 mm wide and located at the wall base. During the 0.92% drift cycle, sliding displacements in the negative direction became substantial, amounting to 15 mm, or 43% of the total lateral drift. This caused the negative-direction compression toe to bear upon the edge of the concrete footing (point b in Figure 28), thereby initiating significant
localized spalling of the footing concrete cover in the vicinity of the wall toe. This caused some asymmetry in the lateral load-displacement hysteretic behaviour in terms of strength and reloading stiffness due to the limited bearing capacity of the negative-direction compression toe. At that stage, out-of-plane displacements, which were slowly increasing since the early stages of the test, reached 3 mm. Face shell spalling first occurred during the 1.29% drift cycle (point e in Figure 28), when the out-of-plane displacements reached 10 mm in magnitude. During the next cycle at 1.47% drift, toe-crushing was first observed when several face shells at the wall toes had deteriorated and spalled off. The exposed vertical reinforcing bars in the toe regions buckled during compression cycles and completely straightened during the subsequent tension cycles. At that stage, out-of-plane displacements reached a considerable magnitude of 30 mm. This value had doubled to 60 mm during the 2.21% drift cycle when both wall toes were severely damaged due to widespread crushing and spalling (point d in Figure 28). During the final 2.58% drift cycle, the out-of-plane displacements reached the critical value of 70 mm (half of the wall thickness) and continued to rapidly increase, leading to global instability of the specimen and the resulting damage shown in the failure photo of (point e in Figure 28). As this failure mechanism occurred after significant toe crushing and face shell spalling, it was characterized as a Class B lateral instability. Figure 31 shows the lateral load-displacement of specimen W2, while Figure 30 summarizes the crack patterns recorded during the test.
Figure 29. Lateral load-displacement hysteresis for specimen W2.
Figure 30. Crack diagrams of the cracking patterns recorded during the test of specimen W2.
4.2.3 Specimen W3

Flexural cracking originated at the wall ends over the bottom five courses during the 0.09% drift cycle and rapidly propagated over the bottom two-thirds of the wall height during the two subsequent drift cycles. At the 0.37% drift cycle, the flexural cracks at the wall base reached a maximum width of 1.25 mm. Wide diagonal cracks originated at the top of the fourth course at both wall ends and progressed in a step-pattern toward mid-length of the wall during the 0.92% drift cycle (point a in Figure 31), reaching a maximum width of 6 mm. The concentration of shear deformation causing this crack pattern can be explained by the absence of horizontal reinforcement between the first and fifth courses. The diagonal shear cracking continued to dominate the response even after the initial signs of face shell spalling initiated at the wall toes during the 1.29% drift.
cycle. At that stage, a sliding plane originated at the top corners of diagonal shear cracks that originated at the top of the fourth wall course and extended along the entire wall length. This plane accommodated significant sliding displacements of over 25 mm during the 1.47% drift cycle; accounting for 45% of the total drift. During the subsequent 1.66% drift cycle, severe spalling and crushing were observed at the bottom course, which then propagated up to the fourth course during consecutive cycles (see point b, point c, and point d in Figure 31). The toe crushing became very severe during the 2.58% drift cycle (point e in Figure 31); however the wall still maintained 75% of its peak lateral load capacity. Finally, the specimen was subjected to a substantial drift of 3.66%. This high drift demand was expected to induce collapse; however, the test was terminated when the maximum displacement range of the out-of-plane supports was exceeded. At the end of the test, the wall still maintained 69% of its peak lateral load capacity, despite significant damage accumulation at both wall toes (point f in Figure 31). Although high drifts were achieved, and the specimen demonstrated overall ductile performance of the specimen, out-of-plane displacements remained insignificant throughout the test. Figure 32 shows the lateral load-displacement of specimen W3, while Figure 33 summarizes the crack patterns recorded during the test.
Figure 32. Lateral load-displacement hysteresis for specimen W3.
Figure 33. Crack diagrams of the cracking patterns recorded during the test of specimen W3.
4.2.4 Specimen W4

Flexural cracking initiated at the bottom course during the 0.09% drift cycle, which extended into the second course during the 0.18% drift cycle. Additional small flexural cracks eventually developed over the bottom six courses during the 0.37% drift cycle (point a in Figure 34). While most flexural cracks remained below 0.2 mm in width, the crack at the base reached a considerable width of 1.25 mm. During the 0.74% drift cycle, the vertical strains became highly concentrated at the wall base with the crack width exceeding 4 mm at that location. During this drift cycle, sliding displacements at the wall base reached a magnitude on the order of 2 mm. During the subsequent drift cycles, sliding displacements increased up to 10 mm in magnitude (point b in Figure 34). The sliding was observed to be very significant during loading in the positive-direction and accounted for up to 40% of the total drift. However, sliding displacements were comparatively

Figure 34. Sequence of key events during the test of specimen W4.
insignificant when the wall was loaded in the negative direction. The behaviour in this direction was dominated by rocking about the negative-direction toe and rigid-body rotation above the wall base. At this stage, the length and width of the flexural cracks above the wall base had mainly stabilized, and the propagation had ceased. This eventually led to the initiation of face shell spalling at the negative-direction toe during the 1.47% drift cycle and a permanent residual displacement in the positive direction at the bottom of the wall (point e in Figure 34). Sliding continued to dominate the response during this cycle and led to a maximum global rotation about the vertical axis on the order of 0.015 radians. The spalling and crushing at the negative-direction wall toe exposed the vertical reinforcing bars, which experienced buckling and straightening action during the subsequent load cycles (point d in Figure 34). At the last 2.21% drift cycle, the permanent residual sliding displacement in the positive direction was 36 mm (0.95% drift); at that point, the flexural crack at the wall base was over 30 mm wide (point e in Figure 34). When the specimen was loaded in the negative-direction during this last cycle, the vertical reinforcing bars fractured at the positive-direction wall toe (see point f in Figure 34). After demolition, significant dowel action damage was observed in the grouted core and the footing, indicating that the bars had been highly strained laterally along the plane of the wall before carrying significant vertical tension, which may have contributed to the eventual fracture. Out-of-plane displacements recorded during the testing were insignificant and primarily caused by the rotational sliding. Figure 35 shows the lateral load-displacement of specimen W4, while Figure 36 summarizes the crack patterns recorded during the test.
Figure 35. Lateral load-displacement hysteresis for specimen W4.
Figure 36. Crack diagrams of the cracking patterns recorded during the test of specimen W4.
4.2.5 Specimen W5

There was extensive flexural cracking that developed in the mortar joints over the lower eight courses of the wall height during the 0.09% drift cycle. These cracks extended over the lower 11 courses during the 0.18% drift cycle, with cracks widths of up to 0.9 mm. During the 0.37% drift cycle, a significant number of diagonal shear cracks developed in a step-pattern over the bottom 15 courses (point a in Figure 34). The same trend continued during the 0.74% drift cycle when a maximum flexural crack width of approximately 7 mm was observed at the base of the negative-direction wall toe. The initiation of face shell spalling was also observed during this cycle; it was characterized by vertical splitting cracks extending through the masonry units up to the third course at both wall toes, as shown in the 0.37% and 0.74% drift photos of point a and b in Figure 37.
respectively. Large flexural cracks up to 10 mm wide were observed at the wall toes during the 0.92% drift cycle (point c Figure 37). The spalling at the wall end zones became significant during the 1.11% drift cycle (see point d in Figure 37), and propagated approximately 400 mm toward mid-length of the wall. The spalling revealed incomplete grouting in the hollow cores at both wall toes (along roughly a 400 mm length) in the bottom two courses of masonry (point e in Figure 37). Subsequent drift cycles significantly exacerbated the spalling and toe-crushing (see point e in Figure 37). During the 1.66% drift cycle, the crushing had propagated approximately 800 mm from both wall toes toward mid-length. At that stage, resistance to gravity and lateral loads relied on about a 1000 mm-long section of relatively intact masonry over the central portion of the wall length. During the 1.84% drift cycle, further disintegration of this central wall section occurred, leading to the out-of-plane collapse of the wall soon after the first load reversal (see point f in Figure 37). The failure was characterized by extensive toe crushing rather than a lateral instability mechanism. Figure 38 shows the lateral load-displacement of specimen W5, while Figure 39 summarizes the crack patterns recorded during the test.
Figure 38. Lateral load-displacement hysteresis for specimen W5.
Figure 39. Crack diagrams of the cracking patterns recorded during the test of specimen W5.
4.2.6 Specimen W6

Specimen W6 was a T-shaped DSRMSW with an asymmetric loading protocol and did not have a sliding restraint at its base. Yielding of the outermost vertical reinforcing bar occurred during the 0.18% ($1.0\Delta y$) drift cycle. During the same drift cycle, a primarily horizontal crack extended over 85% of the wall length with a maximum width of 1.5 mm. During the testing, flexural cracks formed along the horizontal mortar joints over the lower half of wall height in the web end zone. The horizontal flexural cracks formed a triangular fan pattern originating from the web end zone, where the longest cracks developed at the wall base; this corresponded to the curvature demands over the wall height. At the same stage, a 0.1 mm-wide flexural crack formed at the flanged end zone during the loading half-cycle away from the flange. The crack passed through the entire flange thickness and extended 100 mm into the wall web.

Figure 40. Sequence of key events during the test of specimen W6.
The wall reached its maximum capacity during a loading half-cycle toward the flanged end zone at the 1.47% (8.0Δy) drift cycle (point a in Figure 40). This corresponded to the onset of face shell splitting at the flanged end zone, which was followed by sliding penetration of the web through the flanges, with 2.5 mm-wide shear cracks on the outer faces of the flanged boundary elements. At that stage, many of the triangular pattern flexural cracks became inclined, transforming into flexural-shear cracks that passed through masonry face shells toward the base of the web-flange interface. The crack widths continued to increase in a ratcheting-like fashion due to the asymmetric loading protocol; during half-cycles directed toward the flanged end zone, the cracks in the wall web incrementally widened due to increasing tensile strains; however the compressive strains that developed during loading half-cycles directed toward the web end zone were not sufficient to completely close the cracks. Because of this ratcheting behaviour, axial strains within the web-zone remained in net tension throughout each subsequent drift cycle.

Extensive splitting and shear cracking at the flanged end zone initiated face shell spalling during the 1.66% (9.0Δy) drift cycle (point b in Figure 40). At that stage, sliding displacements caused the web to penetrate 9 mm into the flanged boundary elements, thereby contributing to a large concentration of cracking around the web-flange interface at the base of the wall. Flexural crack widths reached a maximum value of 12 mm during this drift cycle, but out-of-plane displacements remained negligible at only 3 mm.

A commonly accepted failure point of 20% strength degradation (the actual value was 14%) relative to the maximum load capacity (point c in Figure 40) occurred during the penultimate loading half-cycle toward the flanged end zone, at 3.42% (18.0Δy) drift. At that stage, the web end zone started to experience minor face shell spalling, whereas the flanged end zone was severely crushed. As a result, the exposed vertical reinforcing bars buckled, and the wall experienced Class
A instability. The bar buckling permitted large compression displacements to develop at the flanged end zone, and sliding began to occur along a wide diagonal shear crack that projected from the flanged end zone at an approximately 35° angle from horizontal. This contributed to a total net sliding displacement of 27 mm; mostly concentrated at the wall base. Despite the maximum flexural crack width surpassing 20 mm magnitude, out-of-plane displacements only reached a magnitude of 11 mm at that stage.

The final load cycle, corresponding to 4.20% drift (23.0Δy) toward the flanged end zone and 0.79% (4.0Δy) drift toward the web end zone, resulting in the complete crushing of the first course of the flanged end zone and a massive 56 mm-wide crack at the base of the web end zone. During this cycle, at 3.42% drift (18.0Δy), fracture of the outer two vertical reinforcing bars at the web end zone took place, resulting in a 43% (71 kN) drop in the lateral load capacity. A similar bar fracture occurred during the specimen W4 test, which also experienced significant sliding and ductility demands before the bar fracture. Global flexural demands on the wall cause axial tensile strain demand in the vertical reinforcement, whereas global sliding demands cause both shear and flexural demands in the vertical reinforcement through dowel action. After several cycles of very high combined flexural and shear demands may cause bar fracture due to low-cyclic fatigue. In the case of specimen W6, bar fracture took place during the 13th post-yield cycle. Since bar fracture prevents the development of the lateral instability mechanism, a loading protocol with fewer cycles at lower ductility demands would have been beneficial for the testing of the specimens.

The maximum out-of-plane displacement of 24 mm during this drift cycle was at least partially due to uneven toe-crushing across the breadth of the flanged end zone. The test was terminated following the 4.21% (23.0Δy) drift cycle when the displacement limits of the out-of-
plane restraints were reached. Figure 49 shows the lateral load-displacement of specimen W6, while Figure 50 summarizes the crack patterns recorded during the test.

Figure 41. Lateral load-displacement hysteresis for specimen W6.
Figure 42. Crack diagrams of the cracking patterns recorded during the test of specimen W6. Note that only the positive direction loading is labelled.
4.2.7 Specimen W7

Figure 43. Sequence of key events during the test of specimen W7.

Specimen W7 was a T-shaped specimen with a symmetric loading protocol and a sliding restraint at the base. A similar crack pattern to specimen W6 was observed at first-yield during the 0.18% (1.0Δy) drift cycle. During the load reversal caused a flexural crack at the wall base to extend over the entire wall length. Subsequent drift cycles created a variation in the crack pattern relative to specimen W6 by causing diagonal “X”-pattern shear cracking toward both end zones as a result of the symmetric loading protocol.

At the 0.92% drift cycle (5.0Δy) (point a in Figure 43) a significant amount of face shell cracking had accumulated at the center of the diagonal “X”-pattern cracking – at approximately mid-length of the wall and over the bottom 20% of its height. Vertical splitting cracks were also initiated at the web end zone during this drift cycle. The maximum in-plane sliding displacement was 3 mm, while the maximum out-of-plane displacement was 11 mm. It was noted that the
maximum out-of-plane displacements occurred when the web end zone was undergoing a tension excursion during the 0.92% drift cycle (5.0Δy); this is different from the typical out-of-plane displacement behaviour, which will be discussed in the following chapters, where the maximum out-of-plane displacements occur when the end zone initiates a compression excursion following the load reversal. This indicated significant eccentric placement of several vertical reinforcing bars near the base of the wall (note that the eccentricity decreased to be nearly negligible at the top of the wall); the bar eccentricities were later confirmed after the bars were exposed following post-face shell spalling and found to range from 10 to 20% of the wall thickness.

The maximum capacity for loading toward the web end zone was reached at the 1.29% (7.0Δy) drift cycle, which coincided with significant face shell splitting and diagonal shear cracking extending through the masonry face shells at the web toes. The maximum capacity for loading toward the flanged end zone took place at drift level of 1.66% drift (9.0Δy) when the face shells at the flanged end zone also began to experience significant splitting. During this drift cycle, the out-of-plane displacements were approximately equal to 15 mm. These displacements more than doubled (32 mm) during the subsequent, 1.84% (10.0Δy), drift cycle (point b in Figure 43). That half-cycle (for loading toward the web end zone) most closely represented the failure point, corresponding to the 20% strength degradation (actually 22%) relative to the maximum load capacity.

During the 1.84% (10.0Δy) drift cycle, further face shell spalling and toe-crushing took place, and a second sliding plane formed at the top of the second course of masonry, creating a 1.5mm slip. Shear cracking at the flanged boundary elements, similar to specimen W6, occurred during the 2.21% (12.0Δy) drift cycle (point c in Figure 43). Moreover, out-of-plane displacements increased to a maximum value of 47 mm at the web end zone, at the top of the third course.
Following this drift cycle, the out-of-plane displacements at the web end zone, which were steadily increasing since the beginning of the test, plateaued at approximately 20 mm during tension excursions; indicating nearly complete straightening of the eccentrically placed web end zone vertical reinforcement.

At the final 3.75% (20.0Δy) drift cycle (point d in Figure 43), severe toe-crushing, and face shell spalling occurred at both wall toes. The exposed vertical reinforcing bars buckled over the bottom two wall courses; this is considered a Class A lateral instability. The out-of-plane displacements reached a significant value of 76 mm during this drift cycle, equivalent to nearly 40% of the wall thickness. This indicates that a Class B instability, generally characterized by the out-of-plane displacements corresponding to 50% of the wall thickness, had nearly occurred; however, the testing had to be terminated when the out-of-plane restraints reached their displacement limits. Figure 52 shows the lateral load-displacement of specimen W7, while Figure 53 summarizes the crack patterns recorded during the test.
Figure 44. Lateral load-displacement hysteresis for specimen W7.
Figure 45. Crack diagrams of the cracking patterns recorded during the test of specimen W7.
4.2.8 Specimen W8

Figure 46. Sequence of key events during the test of specimen W8.

Specimen W8 was a rectangular-shaped specimen with a symmetric loading protocol and a sliding restraint at the base. Yielding of the outermost vertical reinforcing bars occurred during the 0.18% (1.0Δy) drift cycle as flexural cracks formed over the bottom 50% of the wall end zones. During the subsequent drift cycles, the flexural cracking transformed into shear-flexure cracks near the middle of the wall.

At 1.47% (8.0Δy) drift level, significant cracking began to occur through the face shells at both wall toes. This drift cycle also initiated a secondary sliding plane at the top of the second course, above the sliding restraints (a small degree of sliding had already developed at the wall base during prior drift cycles). The maximum lateral load capacity of the wall was reached during the 1.84% (10.0Δy) drift cycle (point a in Figure 46) in both loading directions; this was
accompanied by face shell splitting and spalling and crushing of the grouted core at both wall toes. A maximum crack width of 15.5 mm was observed at the wall base, and sliding displacements at the secondary shear plane increased significantly to approximately 5 mm in each direction. Out-of-plane rotations began pushing small sections of mortar from the horizontal bed joints, although the out-of-plane displacements remained relatively low at 4 mm.

During the 2.21% \((12.0\Delta_y)\) drift cycle (point b in Figure 46), face shell spalling and toe-crushing took place over an area extending 400 mm horizontally from the ends of the wall and 300 mm vertically, i.e. 100 mm into the second masonry course. Class A instability followed at both end zones as the exposed reinforcing bars began to buckle. Load reversal during this drift cycle induced fracture in the outer distributed reinforcing bars in tension, bringing about a 26% drop in the lateral load capacity. During the 2.58% \((14.0\Delta_y)\) drift cycle (point c in Figure 13), which most closely represented the failure point of 20% strength degradation (actually 24%) relative to the maximum load capacity, extensive spalling and crushing area spread over 600 mm horizontally at each wall end. This significant damage limited the effectiveness of the sliding restraint, thus sliding displacements increased up to 25 mm in each loading direction.

During the final, 3.75% \((20.0\Delta_y)\) drift cycle (point d in Figure 46), the toe-crushing damage occurred in a triangular pattern at each wall end zone. It extended 800 mm horizontally from each wall end and 400 mm vertically above the wall base. This crushing pattern effectively localized damage within a distinct plastic hinge region over the two bottom courses of block at the wall base and prevented a further increase of sliding displacements at the wall base. This mechanism governed the pattern of inelastic deformations until the end of the test, which had to be terminated when the displacement limits of the out-of-plane restraints were reached. Figure 55 shows the
lateral load-displacement of specimen W8, while Figure 56 summarizes the crack patterns recorded during the test.

![Lateral load-displacement hysteresis for specimen W8.](image)

Figure 47. Lateral load-displacement hysteresis for specimen W8.
Figure 48. Crack diagrams of the cracking patterns recorded during the test of specimen W8.
4.3 Response of Test Specimens

This section presents recorded and derived data obtained from the experimental testing and provides commentary on the findings. The in-plane response is first presented, followed by the out-of-plane response. A summary of key findings is also subsequently provided. Additional test result numerical data is provided in Appendix B.

4.3.1 In-Plane Response

The in-plane lateral load-displacement hysteresis curves for specimens W1 to W4 are shown in Figure 49, and the hysteresis curves for specimens W5 to W8 are shown in Figure 50. The effect of applied axial precompression is notable when the response of specimen W1 (see Figure 49a) is compared to other specimens, which were tested without precompression (see Figure 49b to Figure 53d and Figure 50). The axial precompression stress increased the lateral load-resisting capacity, but also caused a rapid strength degradation after the ultimate load capacity had been attained. This may be the case for multi-storey residential buildings utilizing DSRMSWs for their SFRS, however this would not be the case for other, more common, applications of DSRMSWs described in Chapter 1. Furthermore, the presence of axial precompression limited the extent of sliding behaviour at the base of the wall; this is evidenced by the pinching behaviour found in all hysteresis curves except for specimen W1.

The influence of the amount of vertical reinforcement is apparent in the hysteretic behaviour of specimen W4 (ρv = 0.15%), which was characterized by the highest h_u/L_w ratio (2.9) of all the specimens. After yielding of vertical reinforcement initiated, the inelastic deformation was concentrated at the wall base in the form of rocking and sliding. The specimen stiffness between first-yield and ultimate capacity was also approximately 80% lower than the other
specimens, which were characterized by lower $h_u/L_w$ ratios. Moreover, the lateral load-resisting
capacity of this specimen was markedly less than the other specimens relative to its wall length
because of its significantly lower vertical reinforcement ratio. The $h_u/L_w$ ratio appeared to have a
less significant effect on the overall specimen behaviour, aside from decreasing the initial elastic
stiffness of specimen W4 relative to the other specimens by 50 to 85% (see Figure 49d).

Specimens W2 and W3 had an equal total area of vertical reinforcement, however
specimen W3 had a larger wall thickness since it utilized 190 mm blocks as opposed to the 140
mm blocks used for specimen W2, thus the vertical reinforcement ratio for W3 ($\rho_v = 0.24\%$) was
27% lower than W2 ($\rho_v = 0.33\%$). As both specimens were designed to exhibit ductile flexural
behaviour, which is controlled by steel yielding and strain hardening, the ultimate load capacities
of the two specimens were very close (within 5%). The larger wall thickness, and hence larger
internal grout column, of specimen W3, may have contributed to increased out-of-plane stability
and a lower rate of strength degradation after the initiation of face shell spalling. This was
demonstrated by the significantly higher displacement ductility and reloading stiffness attained by
specimen W3 compared to specimen W2 (see Figure 49b and Figure 50c).

Accidental incomplete grouting at the toes of specimen W5 became apparent when wide
flexural cracks formed at the wall toe immediately after yielding of the vertical reinforcement took
place. These cracks were followed by rapid face shell spalling and toe-crushing, which
significantly reduced the displacement ductility capacity of the specimen compared to the other
specimens (see Figure 50a). It should be noted that specimens W2 and W5 were of the same design
besides the use of different out-of-plane support arrangements, which did not to appear to influence
the hysteretic behaviour; the differences in response was attributed to two pockets of incomplete
grout in specimen W5 near the base of the wall end zones.
A comparison of hysteresis curves for specimens W6 and W7 illustrates the effects of loading protocol symmetry on the in-plane response of T-shaped DSRMSWs. Aside from the evident response differences for loading in the positive direction, the negative direction responses of the two specimens were similar in terms of peak strength, initial stiffness, and reloading stiffness (see Table 4). However, the specimen W6 experienced a more rapid strength degradation after attaining its maximum lateral load capacity than specimen W7, where strength degradation is defined as $\Delta u / \Delta 0.8u$ (see Table 4). This can be explained by the difference in the magnitude of sliding displacements between the specimens. The sliding restraints for specimen W7 acted to significantly reduce sliding at higher ductility levels, while specimen W6 was able to slide freely throughout the test. This was found to have helped distribute inelastic deformations over the bottom three wall courses of specimen W7, whereas a comparatively high degree of inelastic deformation was concentrated at the bottom course of specimen W6. The sliding displacements also are believed to have contributed to the fracture of vertical reinforcing bars in specimen W6 (described in the previous section). This fracture caused a 43% (71 kN) drop in the lateral load capacity of specimen W6.

Note that the average reloading stiffness of specimen W6 was 8.5 times higher than that of W7 in the positive direction. This contributed to specimen W6 having a 105% (3.4 kN·m) elevated level of energy dissipation compared to specimen W7 (the amount of energy dissipation was determined as the area under the load-displacement curve, in the top-left quadrant of the lateral load-displacement hysteresis plot). This can be attributed to the ratcheting action observed at the web end zone of specimen W6 when it was subjected to compression. Due to the asymmetric loading protocol, the axial compressive stresses within the web end zone of specimen W6 were much lower and contributed to avoiding toe-crushing and face shell spalling in the specimen. Thus,
the plastic tensile strains in the vertical reinforcement accumulated and the bars remained laterally supported by the surrounding masonry throughout the compression excursions, which prevented bar buckling and the associated low-stiffness response to vertical compressive displacements. Furthermore, as the cracks opened and closed, mismatches between the top and bottom crack surfaces developed under increasing shear deformations. As a result, these cracks remained partially open during the entire cyclic loading because the vertical reinforcement and aggregate-interlock action proved sufficient to transfer the relatively low applied compression stresses across the crack, without the need for complete crack closure to transfer these compression stresses.

A comparison of the hysteresis curves for T-shaped specimen W7 versus the rectangular-shaped specimen W8 reveals the effect of flanged boundary elements on the lateral load-displacement response of these specimens. The most apparent difference is that specimen W7 exhibited a 59% (132 kN) increase in maximum strength during positive-direction loading compared to specimen W8. The flanged end zone of specimen W8 had twice the amount of vertical reinforcement compared to the specimen W8 end zone; this resulted in the significantly higher maximum lateral load capacity of specimen W7 for negative-direction loading. Despite the difference in maximum lateral load capacities, specimen W8 experienced very similar strength degradation compared to specimen W7 for negative-direction loading, as shown in Table 4. The residual strength of specimen W8 was however 13% (18 kN) lower than specimen W7, primarily due to the compression-controlled flexural response of specimen W8 for loading toward the web end zone, which was characterized by the significant face shell spalling and toe-crushing in the post-maximum strength stage. The maximum strength due to negative-direction loading was also higher for specimen W7 compared to specimen W8 due to the presence of the flange in specimen W7, which resulted in a larger compression zone and an extended moment arm for flexural
resistance. However, because the load capacity of specimen W8 was tension-controlled in the negative direction, the difference between the maximum strengths was only 10% (23 kN).

The average negative-direction reloading stiffness of specimen W7 was approximately twice that of W8. This can be attributed to the presence of the flanged end zone in specimen W7, which offered out-of-plane restraint and stiffness that was not available for rectangular-shaped specimen W8. As a result, specimen W7 maintained its out-of-plane alignment throughout the test. In comparison, it appears that the out-of-plane displacements caused a more rapid toe-crushing and face shell spalling at both end zones in specimen W8.
4.3.1.1 In-Plane Lateral Load-Displacement Backbone Curves

Table 3 summarizes the idealized lateral load-displacement backbone curves for each of the specimens with reference to Figure 51, which presents an idealized backbone curve based on points \( c, y, u, d, \) and \( e \). The backbone points represent separate instances in the hysteretic behaviour of the specimens during the testing. Point \( c \) represents the instance of first cracking of the masonry at the tension end zone; point \( y \) represents the instance of first-yield of the outermost vertical reinforcing bars at the tension end zone; point \( u \) represents the instance of the attainment of the ultimate lateral load capacity; point \( d \) represents the point of 20% strength degradation from point \( y \); and point \( e \) represents the instance of the residual strength for the specimen at the termination of the test.

Analysis of Table 3 shows that the relation between \( Q_c \) and \( Q_y \) is strong among the specimens; \( Q_c \) was found to be approximately 66% of \( Q_y \), with a standard deviation (STD) of ±10% and a coefficient of variation (COV) of 15.9%. The relation between \( Q_u \) and \( Q_y \) was also found to be relatively strong among the specimens, with \( Q_u \) being approximately 31% greater than \( Q_y \) and an STD of ±16% and a COV of 12.3%. As can then be expected, the relation between \( Q_d \) and \( Q_y \) is also quite strong, with \( Q_d \) being just 5% larger than \( Q_y \), with an STD of 13± and a COV of 12.1%. \( Q_m \) is understandably not well correlated to \( Q_y \) since \( Q_m \) is highly dependent on the extent to which the specimens were loaded beyond yield, which typically differed among the specimens due to differing governing failure mechanisms and the displacement capacity limitations of the test setups in terms.
Figure 49. Lateral load-displacement behaviour of specimens: a) W1, b) W2, c) W3, and d) W4. Backbone curve shown in red (see Table 3).
Figure 50. Lateral load-displacement behaviour of specimens: a) W5, b) W6, c) W7, and d) W8. Backbone curve shown in red (see Table 3).
As shown in the righthand columns of Table 3, the lateral drift relations were not as well-correlated as the lateral load relations; the COVs of the $D_c/D_y$, $D_u/D_y$, $D_d/D_y$, and $D_m/D_y$ relationships all exceeded 39.9%. This can be partly explained by the fact some specimens demonstrated significantly higher sliding displacements compared to other specimens. Sliding was found to increase the specimen ductility while not significantly affecting the strength. As a result, specimens W4, W6, W7, and W8 had much higher $D_u/D_y$ values relative to the W1, W2, W3, and W5 specimens. Furthermore, specimens that experienced higher rates of strength degradation, such as specimen W5, which was affected by zones of partial grouting, or specimen W1, which was loaded in precompression, had much lower and $D_d/D_y$ values. Interestingly, despite a large COV for $D_y$ among the specimens, the mean value of 0.20% agrees very well with past findings on RMSWs in general based on experimental studies by Paulay and Priestley [12].

![Structural behaviour at specified backbone points](image)

- **c**: First cracking
- **y**: First yielding of vertical reinforcement
- **u**: Ultimate load capacity
- **d**: 80% of ultimate load capacity
- **m**: Maximum displacement ductility

Figure 51. Backbone envelope based on lateral load-displacement hystereses of Figure 49 and Figure 50.
Table 3. Comparison of lateral load - displacement hystereses.

<table>
<thead>
<tr>
<th>ID</th>
<th>Loading Direction</th>
<th>Lateral Load (kN)</th>
<th>Lateral Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(Q_c/Q_y)</td>
<td>(Q_y)</td>
</tr>
<tr>
<td>W1</td>
<td>1</td>
<td>0.62</td>
<td>199</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.68</td>
<td>184</td>
</tr>
<tr>
<td>W2</td>
<td>1</td>
<td>0.54</td>
<td>185</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.44</td>
<td>170</td>
</tr>
<tr>
<td>W3</td>
<td>1</td>
<td>0.70</td>
<td>155</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.73</td>
<td>132</td>
</tr>
<tr>
<td>W4</td>
<td>1</td>
<td>0.57</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.63</td>
<td>27</td>
</tr>
<tr>
<td>W5</td>
<td>1</td>
<td>0.59</td>
<td>187</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.58</td>
<td>159</td>
</tr>
<tr>
<td>W6</td>
<td>1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.72</td>
<td>183</td>
</tr>
<tr>
<td>W7</td>
<td>1</td>
<td>0.73</td>
<td>250</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.71</td>
<td>152</td>
</tr>
<tr>
<td>W8</td>
<td>1</td>
<td>0.87</td>
<td>157</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.79</td>
<td>149</td>
</tr>
</tbody>
</table>

| MEAN | 0.66 | 154.9 kN | 1.31 | 1.05 | 0.76 | 0.38 | 0.20 % | 6.09 | 10.40 | 15.07 |
| STD  | 0.10 | 55.4 kN  | 0.16 | 0.13 | 0.34 | 0.15 | 0.08 % | 3.99 | 5.79  | 9.06  |
| COV (%) | 15.9 | 35.8 | 12.3 | 12.1 | 44.8 | 39.9 | 42.9 | 65.6 | 55.7 | 60.1 |
4.3.1.2 In-Plane Ductility Characteristics

Ductility relates to the capacity of a structure to undergo inelastic displacements and damage prior to failure. Ductility characteristics for the test specimens are relevant for evaluating their response in the post-yielding phase. Figure 52 describes the methodology for idealizing lateral load-displacement response of the test specimens in the form of idealized elastic-perfectly-plastic (EPP) envelopes using an equal-energy-based approach initially proposed by Tomazevic [58]. These EPP curves are shown as red lines in Figure 53 and Figure 54. The key parameters used in the calculations, as well as the results, are summarized in Table 4 for both loading directions for each of the tests. The plastic lateral capacity, $V_{EPP}$, can be computed from the experimentally-obtained force-displacement envelope provided in equation (1).

$$V_{EPP} = K_e \times (\Delta_{max} - (\Delta_{max}^2 - \frac{2A_{env}}{K_e})^{\frac{1}{2}})$$  (1)

Where $A_{env}$ represents the area below the experimental resistance envelope in terms of kN·m.

![Figure 52. Methodology for idealizing force-displacement response.](image)
### Table 4. Idealized elastic-perfectly-plastic (EPP) envelope parameters

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Loading Direction</th>
<th>$V_y$ (kN)</th>
<th>$V_u$ (mm)</th>
<th>$V_{EPP}$ (kN)</th>
<th>$R (\frac{V_{EPP}}{V_u})$</th>
<th>$\Delta_y$ (mm)</th>
<th>$\Delta_e$ (mm)</th>
<th>$\Delta_{EPP}$ (mm)</th>
<th>$\Delta_{max}$ (mm)</th>
<th>$K_y = K_e$ (kN/m)</th>
<th>$A_{env}$ (kN-m)</th>
<th>$\mu_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1</td>
<td>(+)</td>
<td>199</td>
<td>250</td>
<td>211</td>
<td>0.84</td>
<td>8.3</td>
<td>8.8</td>
<td>20</td>
<td>30</td>
<td>24034</td>
<td>5.5</td>
<td>2.3</td>
</tr>
<tr>
<td></td>
<td>(-)</td>
<td>184</td>
<td>249</td>
<td>232</td>
<td>0.93</td>
<td>8.3</td>
<td>10.4</td>
<td>26</td>
<td>29</td>
<td>22222</td>
<td>5.4</td>
<td>2.5</td>
</tr>
<tr>
<td>W2</td>
<td>(+)</td>
<td>185</td>
<td>217</td>
<td>176</td>
<td>0.81</td>
<td>12.2</td>
<td>11.7</td>
<td>66</td>
<td>97</td>
<td>15114</td>
<td>16.1</td>
<td>5.7</td>
</tr>
<tr>
<td></td>
<td>(-)</td>
<td>170</td>
<td>199</td>
<td>165</td>
<td>0.83</td>
<td>10.4</td>
<td>10.1</td>
<td>56</td>
<td>78</td>
<td>16284</td>
<td>12.0</td>
<td>5.5</td>
</tr>
<tr>
<td>W3</td>
<td>(+)</td>
<td>155</td>
<td>206</td>
<td>154</td>
<td>0.75</td>
<td>4.2</td>
<td>4.2</td>
<td>74</td>
<td>150</td>
<td>37081</td>
<td>22.9</td>
<td>17.8</td>
</tr>
<tr>
<td></td>
<td>(-)</td>
<td>132</td>
<td>183</td>
<td>155</td>
<td>0.85</td>
<td>7.6</td>
<td>8.9</td>
<td>80</td>
<td>98</td>
<td>17368</td>
<td>14.6</td>
<td>9.0</td>
</tr>
<tr>
<td>W4</td>
<td>(+)</td>
<td>35</td>
<td>43</td>
<td>42</td>
<td>0.98</td>
<td>5.3</td>
<td>6.4</td>
<td>65</td>
<td>77</td>
<td>6579</td>
<td>3.1</td>
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<td></td>
<td>(-)</td>
<td>27</td>
<td>34</td>
<td>32</td>
<td>0.94</td>
<td>3.0</td>
<td>3.6</td>
<td>49</td>
<td>59</td>
<td>8882</td>
<td>1.8</td>
<td>13.6</td>
</tr>
<tr>
<td>W5</td>
<td>(+)</td>
<td>187</td>
<td>188</td>
<td>152</td>
<td>0.81</td>
<td>6.8</td>
<td>5.6</td>
<td>20</td>
<td>56</td>
<td>27339</td>
<td>8.1</td>
<td>3.6</td>
</tr>
<tr>
<td></td>
<td>(-)</td>
<td>159</td>
<td>181</td>
<td>148</td>
<td>0.82</td>
<td>14.8</td>
<td>13.8</td>
<td>46</td>
<td>67</td>
<td>10729</td>
<td>8.9</td>
<td>3.3</td>
</tr>
<tr>
<td>W6</td>
<td>(+)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>(-)</td>
<td>186</td>
<td>241</td>
<td>217</td>
<td>0.90</td>
<td>6.7</td>
<td>7.8</td>
<td>68</td>
<td>123</td>
<td>27761</td>
<td>25.8</td>
<td>8.7</td>
</tr>
<tr>
<td>W7</td>
<td>(+)</td>
<td>249</td>
<td>355</td>
<td>274</td>
<td>0.77</td>
<td>5.8</td>
<td>6.4</td>
<td>86</td>
<td>127</td>
<td>42931</td>
<td>33.9</td>
<td>13.5</td>
</tr>
<tr>
<td></td>
<td>(-)</td>
<td>152</td>
<td>253</td>
<td>235</td>
<td>0.93</td>
<td>6.9</td>
<td>10.7</td>
<td>114</td>
<td>133</td>
<td>22029</td>
<td>30.0</td>
<td>10.7</td>
</tr>
<tr>
<td>W8</td>
<td>(+)</td>
<td>158</td>
<td>225</td>
<td>194</td>
<td>0.86</td>
<td>5.4</td>
<td>6.6</td>
<td>76</td>
<td>130</td>
<td>29259</td>
<td>24.5</td>
<td>11.5</td>
</tr>
<tr>
<td></td>
<td>(-)</td>
<td>161</td>
<td>231</td>
<td>197</td>
<td>0.85</td>
<td>5.6</td>
<td>6.8</td>
<td>88</td>
<td>127</td>
<td>28750</td>
<td>24.3</td>
<td>12.9</td>
</tr>
</tbody>
</table>
Figure 55 provides a visual comparison of the idealized EPP curves. The effective stiffness of the first slope of the EPP curve, $K_e$, is equal to $K_y$ (effective stiffness at first yield), which links the lateral load and displacement at the point of first-yield of the outermost vertical reinforcing bars, i.e. $V_y$ and $\Delta_y$, respectively. $V_u$ is defined as the maximum lateral load recorded during the experimental test. $\Delta_{max}$ is the maximum displacement attained during the test, except for specimens W4 and W6, where $\Delta_{max}$ refers to the maximum displacement during the cycle before bar fractures that caused dramatic drops in the lateral load near the end of the test and would thus distort the accuracy of the EPP curve. The EPP curve data for specimens W1 to W5, W7, and W8 are calculated in both directions, but only in the negative direction for specimen W6 due to the asymmetric loading protocol (the positive direction does not yield meaningful results). As shown in Table 4, the ratio $R$ (equal to $V_{EPP} / V_u$) ranges from 0.75 to 0.98 between the specimens, where the highest values correspond to walls experiencing high sliding displacements and for loading toward the flanged end zones of the T-shaped specimens. The lowest $R$ values corresponded to loading toward the web end zone of specimen W6 and for the positive direction loading of specimen W3, which had very high drift cycles near the end of the test. Loading toward the end zones of the rectangular specimen produced an intermediate value of 0.87.
Figure 53. Lateral load-displacement behaviour of specimens: a) W1, b) W2, c) W3, and d) W4. Elastic-perfectly plastic curves (EPP) shown in red.
Figure 54. Lateral load-displacement behaviour of specimens: a) W5, b) W6, c) W7, and d) W8. Elastic-perfectly plastic curves (EPP) shown in red.
Figure 55. Summary of elastic-perfectly plastic curves (EPP) for all specimens.

The point of 20% strength degradation (corresponding to $0.8V_u$) was selected as an indication for the designated failure point (a commonly accepted value for failure); hence $\Delta_{EPP}$ is defined as the lateral displacement where the horizontal $0.8V_u$ line intersects the experimental envelope. The ultimate ductility factor, $\mu_u$, is defined as $\Delta_{EPP} / \Delta_e$, where $\Delta_e = \Delta_{EPP}/K_e$ [58]. The calculated $\mu_u$ values for the test specimens, ranging from 2.3 to 17.8, are inverse to the $R$ values. As noted by Tomazevic [58], experimental results such as these often indicate much larger ductility values than is acceptable for practical masonry construction. The lowest $\mu_u$ values correspond to specimen W1, which was loaded with precompression; specimen W5, which was affected by partial grouting and failed prematurely; specimen W2, which experienced out-of-plane
displacements that affected the ductility range of the wall; specimen W6 for loading toward the flanged end zones of the T-shaped specimen. The highest values corresponded to specimens W3, W4, W7, and W8, which had lower h_u/t_w ratios. Based on specimen W7, it also appeared that loading toward the web end zone produced higher overall \( \mu_u \) values. The total average \( \mu_u \) value was determined to be 9.6 for the rectangular specimens (except W5, which was excluded due to the partial grouting) and 11.0 for the T-shaped specimens (averaging between the scenarios of loading toward and away from the flanged end zone).

### 4.3.2 Out-of-Plane Response

A description of the mechanics of the different classes of lateral instability is provided in this section to help describe the numerical results of the out-of-plane response of the wall specimens. As mentioned earlier, Class C lateral instability can lead to unexpected premature failure and collapse of slender flexure-controlled RCSWs and DSRMSWs. The first mechanical model describing this lateral instability mechanism in structural walls was hypothesized by Paulay and Priestley [23]. Subsequent refinements to the model specifically focused on RCSWs were proposed by Chai and Elayer [59] and similar improvements focused on uniaxial prismatic RM specimens were proposed by Azimikor et al. [35]. Despite the development of these models, lateral instability behaviour has been observed in only a few of past experimental studies on DSRMSWs ([12], [13]). In these studies, the observed out-of-plane instabilities were always preceded by face shell spalling or toe crushing, and hence should generally be classified as Class B instabilities.

Both Class B and C out-of-plane instabilities are inherently related in that they both depend on out-of-plane rotations and displacements, and they exhibit similar behaviour over the plastic hinge height at the end zones of the wall. Figure 56 describes the axial strain versus out-of-plane
displacement component recorded at mid-height of the plastic hinge based on the numerical results for DSRMSW specimens W2 and W7; this is similar to out-of-plane displacements recorded at mid-height of uniaxial prismatic specimens tested by Azimikor et al. [35]. When a wall end zone is subjected to relatively high tensile strains (point 1), a relatively uniform tensile crack pattern develops along the mortar joints over the plastic hinge height. During load reversal (point 2), increasing compression stresses in the wall end zone are resisted solely by the vertical reinforcing bars extending across the open horizontal cracks. This may cause the end zone to experience out-of-plane displacements due to eccentric bar placement and load path, even at minimal compression stress levels. At this stage, two mechanisms of response with different consequences are possible (denoted by paths to points 3a and 3b, respectively). The first mechanism (point 3a) develops when the flexural cracks close before out-of-plane displacements attain a critical value (theoretically half of the wall thickness). Crack closure creates a restoring transverse bending moment due to internal compressive forces in the masonry face shells at the points of contact. This restores the lateral stiffness of the end zone and returns it to a position close to plumb near the time of load reversal (point 4). Some residual displacements may remain due to misalignments between masonry units because the cracks had closed while the end zone was still displaced out-of-plane. The second mechanism (point 3b) develops when the flexural cracks do not close before the critical out-of-plane displacement being attained. In this case, the out-of-plane displacements will continue to increase, and lateral instability may ensue.
Figure 56. A cycle of average axial strain versus out-of-plane displacement in an end zone of a DSRMSW subjected to in-plane reversed cyclic loading. Note that the out-of-plane displacement is recorded at half of the plastic hinge height.

If lateral instability does not take place (path toward point 3a) and the specimen's vertical alignment is nearly restored (point 4), out-of-plane displacements may increase by a small amount during load reversal; this behaviour is caused by elastic compressive strain recovery in the vertical reinforcing bars which deformed eccentrically during the initial crack closure (point 5). However,
the displacements are expected to steadily decline as the end zone is subjected to high tensile stresses (point 6), and eventually drop to approximately zero value when the wall end zone returns to its original position in tension (point 1).

Class B and C instabilities have a very similar effect on a wall end-zone; the primary difference between the instability types is the coordinates of point 4. Since a Class B instability includes face shell spalling and toe-crushing, one may expect the axial compressive strain at point 4 to be higher than the strain corresponding to a Class C instability. Furthermore, the location of the restorative forces brought about by crack closure is different for these two mechanisms. For Class C instability, the crack closure occurs within the end zone, while for a Class B instability, the crack closure occurs immediately adjacent to the crushed or spalled region of the end zone (toward mid-length of the wall). As toe crushing and face shell spalling extend from the wall end toward mid-length, the magnitude of the residual out-of-plane displacement at point 4 is therefore expected to be more significant.

The mechanics of lateral instability development described above can also be applied to end zones of slender RCSWs ([23], [59]). The primary difference is that slender RCSWs generally have higher amounts of vertical and confinement reinforcement within the end zones and employ two layers (curtains) of vertical reinforcement along the wall length. Vertical reinforcement layers in slender RCSWs are typically placed near the wall faces, whereas DSRMSWs usually have a single layer of vertical reinforcement placed along the wall centerline. The typically higher vertical reinforcement content in the end zones of an RCSW will result in lower critical out-of-plane displacements, thus generally decreasing the out-of-plane displacements necessary to precipitate point 3b [23]. The higher vertical reinforcement content will also provide additional shear stiffness across the open cracks, thus reducing the relatively small out-of-plane displacements present at
points 4 and 5. End zones of slender RCSW also generally incorporate confinement reinforcement, which permits these end zones to experience higher compressive strains before toe-crushing than slender RMSW end zones. Therefore, for Class C instability, compressive strains at the end zones of an RCSW (points 4 and 5 of the mechanism) are expected to reach higher levels than the corresponding strains in end zones of a DSRMSW.

The effect of vertical reinforcement content on out-of-plane displacements that lead to instability in DSRMSWs and RCSWs was quantified by Paulay and Priestley [23]. They proposed equation (2) for estimating the critical out-of-plane displacement (normalized to the wall thickness, t) prior to lateral instability, $\xi_{\text{critical}}$, which is dependent on the vertical reinforcement content and the mechanical properties of masonry and steel:

$$\xi_{\text{critical}} = 0.5(1 + 2.35m - \sqrt{5.53m^2 + 4.7m})$$ (2)

where the mechanical reinforcement ratio, $m$, is defined as:

$$m = \frac{A_s \times f_y}{A_{m/c} \times f'_{m/c}}$$

Where $A_s$ is the area of vertical reinforcement within the wall end zone, $A_{m/c}$ is the masonry/concrete end zone area (for DSRMSWs and RCSWs, respectively), $f_y$ is the steel yield strength, and $f'_{m/c}$ is the masonry/concrete compressive strength (for DSRMSWs and RCSWs, respectively).

As the mechanical reinforcement ratio increases, generally due to increased end zone vertical reinforcement content, the critical out-of-plane displacement for lateral instability decreases. For example, a DSRMSW with a typical end zone vertical reinforcement layout (15M bars spaced at 400 mm per layer, similar to the specimens tested in this experimental study)
resulting in an $A_s = 400 \text{ mm}^2$, steel yield strength $f_y = 400 \text{ MPa}$, and a masonry compressive strength of $f_m' = 25 \text{ MPa}$, the resulting $\xi_{critical}$ value is 0.27 for one layer of reinforcement or 0.21 for two layers of reinforcement.

4.3.2.1 Axial Strain vs. Out-of-Plane Displacement Responses

Based on the previous discussion, it may be concluded that axial strains play a significant role in the development of both Class B and C out-of-plane instabilities in DSRMSWs. However, Figure 57 and Figure 58, which provide visual comparisons of the experimentally recorded axial strains averaged over the full wall height to the out-of-plane displacements recorded at approximately mid-height of the plastic hinge region (600 mm above the wall base) for specimens W1 to W4 and W5 to W8, respectively, reveals an inconsistent relation between the two measures.

When studying the averaged strains shown in Figure 57 and Figure 58, it is essential to note that these figures neglect the concentration of tensile strains over the plastic hinge height, which were found to be 80% larger than the averaged strains typically. While this tensile strain concentration is neglected, average tensile strains were used as a defining parameter in past studies on uniaxially loaded prismatic specimens, and hence the average tensile strain values are presented here for comparison with those studies. The plastic hinge height was generally in the range between one-quarter of the wall length to the three-quarters of the wall length. Moreover, these averaged strains were plotted in place of the plastic hinge strains because the strain recording points located in the lower portions of the wall (in the plastic hinge region) were often damaged much earlier in the testing than the strain recording points which were located in the upper portions of the wall. Therefore, it is worth noting that the tensile strains over the plastic hinge height are expected to be significantly higher than those shown in Figure 57 and Figure 58.
Figure 57. Averaged end zone axial strain versus out-of-plane displacement (reference point at the top of the third course of masonry): a) W1, b) W2, c) W3, and d) W4.
Figure 58. Averaged end zone axial strain versus out-of-plane displacement (reference point at the top of the third course of masonry): e) W5, f) W6, g) W7, and h) W8.
It was observed that cracking over the plastic hinge height was not always uniform; some of the mortar joints in the lower portion of the wall remained uncracked throughout the test. For instance, in specimens W1, W3, and W4 the mortar joint above the first course of masonry to stay uncracked. This appeared to have hindered the development of out-of-plane rotations large enough to precipitate instability. The lack of cracking at that location may be attributed to the grout at the wall toes generally being of somewhat inferior quality; this was potentially due to the possible presence of mortar droppings accumulated here during construction, as well as a potentially higher water content and lower consolidation of the grout placed in these blocks during pumping. These characteristics of the grout near the base of the wall can be expected in construction practice – the masons were instructed to perform construction using the same techniques they would commonly employ on site. Although not explicitly tested, the somewhat inferior quality grout was likely characterized by lower bond strength to the vertical reinforcement; thus, strain penetration could be expected to be more significant in lower portions of the wall. Furthermore, upon the occurrence of face shell spalling and toe-crushing, which exposed the vertical reinforcing bars at the wall toes, high compressive and tensile strains would become highly localized. This caused existing cracks in the upper portions of the wall to maintain a nearly constant width in all subsequent drift cycles, indicating a lack of uniformity in the strains over the plastic hinge height as well.

Taking this into consideration and inspecting Table 5, which summarizes the in-plane and out-of-plane deformations at the critical testing stages, only specimens W2 and W7 experienced significant out-of-plane displacements, whereas the other specimens experienced only relatively small out-of-plane displacements. In fact, while all specimens tested during this experimental study experienced some amount of out-of-plane displacement, only specimens W2 experienced
displacements sufficiently large enough to eventually culminate in lateral instability (Class B) (note that although large out-of-plane displacements were recorded during the testing of specimen W7, these displacements did not culminate in lateral instability). Even at more modest axial tensile strains of up to 0.005, the end zone out-of-plane behaviour of specimens W2 and W7 reacted significantly more strongly than the other specimens (although specimen W6 also experienced rapid increases of out-of-plane displacement early in the testing, these somewhat stabilized as the test progressed). This indicates the importance of other factors affecting the out-of-plane response of the DSRMSW specimens.

The large sliding displacements experienced by the unrestrained specimens W4 and W6 were observed to create a tendency for the reinforcing bars crossing the primary sliding plane (located at the wall base) to rotate and deform in the plane of the wall rather than out-of-plane; this significantly reduced the tendency of the end zone to develop significant out-of-plane rotations and displacements. Despite the asymmetric loading protocol producing high axial strains at the web end zone before toe-crushing for specimen W6, the sliding displacements were found to be a primary factor inhibiting Class C instability from occurring in specimen W6. Figure 57d and Figure 58b illustrate the magnitudes of out-of-plane displacements for specimens W4 and W6, respectively.

The low compression loading at the web end zone of specimen W6 also failed to fully close the open flexural cracks over the plastic hinge height. This created relatively rough crack surfaces compared to specimen W7, which had a symmetric loading protocol. It appeared that these uneven crack surfaces also contributed to decreased out-of-plane rotations in the specimen W7 by reducing the clear space across the crack, thus limiting out-of-plane curvatures prior to face shell contact being established.
Table 5. Measured in-plane and out-of-plane deformations (note that only negative-direction loading is examined for the T-shaped specimens, W6 and W7)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Loading Direction</th>
<th>During the maximum load drift cycle</th>
<th>During the 20% strength degradation drift cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>In-plane</td>
<td>Out-of-plane</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Δ_u</td>
<td>d_u</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(mm)</td>
<td>(%)</td>
</tr>
<tr>
<td>W1</td>
<td>(+)ve</td>
<td>14</td>
<td>0.38</td>
</tr>
<tr>
<td></td>
<td>(-)ve</td>
<td>18</td>
<td>0.49</td>
</tr>
<tr>
<td>W2</td>
<td>(+)ve</td>
<td>46</td>
<td>1.27</td>
</tr>
<tr>
<td></td>
<td>(-)ve</td>
<td>24</td>
<td>0.66</td>
</tr>
<tr>
<td>W3</td>
<td>(+)ve</td>
<td>23</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td>(-)ve</td>
<td>48</td>
<td>1.26</td>
</tr>
<tr>
<td>W4</td>
<td>(+)ve</td>
<td>39</td>
<td>1.03</td>
</tr>
<tr>
<td></td>
<td>(-)ve</td>
<td>50</td>
<td>1.31</td>
</tr>
<tr>
<td>W5</td>
<td>(+)ve</td>
<td>7</td>
<td>0.19</td>
</tr>
<tr>
<td></td>
<td>(-)ve</td>
<td>28</td>
<td>0.74</td>
</tr>
<tr>
<td>W6</td>
<td>(+)ve</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>(-)ve</td>
<td>45</td>
<td>1.18</td>
</tr>
<tr>
<td>W7</td>
<td>(+)ve</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>(-)ve</td>
<td>64</td>
<td>1.68</td>
</tr>
<tr>
<td>W8</td>
<td>(+)ve</td>
<td>49</td>
<td>1.29</td>
</tr>
<tr>
<td></td>
<td>(-)ve</td>
<td>50</td>
<td>1.32</td>
</tr>
</tbody>
</table>
Furthermore, it was observed that an eccentric placement of the vertical reinforcing bars also acted to reduce overall out-of-plane displacements. This was particularly evident during the testing of specimen W7, which had significant out-of-plane displacements on both sides of the wall. When the web end zone was loaded in tension, out-of-plane displacements exceeded 10% of the wall thickness, as shown in Figure 58c. However, when the web end zone was transitioning from tension to compression, out-of-plane displacements reached nearly 40% of the wall thickness, but this occurred in the opposite transverse direction relative to the out-of-plane displacements developed when the end zone was in tension. The net out-of-plane displacement, therefore, exceeded the critical value of 50% of the wall thickness prior to lateral instability, but the distribution of out-of-plane displacements during the transition from tension to compression allowed equilibrium to be maintained throughout the loading protocol drift cycles. It also appeared that the eccentric bar placement caused the crack closure mechanism to act more rapidly than in the otherwise similar specimen W6, which had a relatively concentric alignment of vertical reinforcing bars with regards to the vertical wall centerline.

The flanged boundary elements had a noticeable effect on the out-of-plane response of the wall end zones. It was apparent that the increased lateral strength and stiffness of the flanged end zones prevented both Class B and C instability from initializing at the flanged end zones of specimens W6 and W7. However, the bar buckling associated with Class A instability was not inhibited by the flanged boundary elements following toe-crushing at high ductility demands.

It can be seen from Figure 58c (specimen W7) that the point of maximum out-of-plane displacement for each drift cycle, i.e. point 3a in Figure 56, initially occurred at low levels of axial tensile strain (approximately 0.2%), but progressively took place at increasing levels of tensile strain until the initiation of face shell spalling and toe-crushing at the web end zone. The strain at
point 3a was related to the peak strain, which was experienced by the end zone during the cycle, i.e. point 1 in Figure 56. For lower drift demands, the axial strain at point 3a was approximately 30-40% of the strain at point 1; at higher drift demands, this point 3a / point 1 strain relation was between 50-60%. Similarly, the residual out-of-plane displacement corresponding to the point of maximum compressive strain, i.e. point 4 in Figure 56, also increased with each load cycle as the crack closure mechanism was initiated at progressively increasing out-of-plane displacement.

The out-of-plane displacements of specimen W7 were observed to increase most rapidly before the initiation of toe-crushing, which is indicated by the first instance of a negative average axial strain of approximately 0.1% in Figure 58c. After the onset of toe-crushing, the out-of-plane displacements continued to increase, although at a slower rate. This was a result of relatively flexible exposed vertical reinforcing bars which continued to move laterally out-of-plane before they experienced buckling, i.e. a Class A instability.

4.3.2.2 Key Parameters Influencing the Out-of-Plane Displacement Responses

While all the wall specimens tested in this experimental study were designed to experience lateral instability, only two specimens experienced a type of global lateral instability failure, and even this was a Class B instability rather than the more concerning Class C instability. These results confirm the challenges associated with achieving lateral instability in DSRMSWs and explain the absence of experimental and actual earthquake evidence related to this phenomenon. However, based on the experimental results from this study and contributions by other researchers, a number of factors were found to have a significant influence upon the development (or absence thereof) of lateral instability mechanisms in DSRMSWs, as discussed in this section:
1. **Wall $h_u/t_w$ ratio:** This is currently the only indicator of lateral instability recognized by CSA S304-14 [17]. All specimens were designed with $h_u/t_w$ ratios that were between 32% to 79% greater than the maximum CSA S304-14 [17] limits for DSRMSWs; however, none of the specimens experienced a Class C instability and only specimen W2, with $h_u/t_w$ ratio of 27.1, experienced a Class B instability. The results of this limited study, therefore, indicate that the current CSA S304-14 [17] $h_u/t_w$ limits could be potentially overly conservative. More importantly, the use of $h_u/t_w$ limits alone to assess the vulnerability of DSRMSWs to lateral instability may be inappropriate - other factors discussed below may also need to be considered. Further experimental testing or numerical modelling is required to confirm the findings from the specimens presented in this thesis.

2. **Ductility and tensile strain demand effects:** Based on the results of the tests on specimens W2, tensile strains higher than 5-10 times the steel yield strain $\varepsilon_y$ (i.e. between 1% and 2% strain), were required to precipitate large out-of-plane displacements in the specimens, which had $h_u/t_w$ ratio of 27 and 21, respectively, and even these large strains did not always cause lateral instability. Such high tensile strains may be expected at relatively large displacement ductility levels and were more than 7 when instability occurred in specimen W2, and more than 9 for specimen W7 before toe-crushing prevented lateral instability in this specimen. Ductility demands experienced by the wall specimens generally significantly exceeded the ductility capacity requirements set forth by NBCC 2015 [18] for ductile DSRMSW classes. Moreover, since tensile strain demands are highly related to the plastic hinge height, which is in-turn highly related to the $h_{eff}/L$ ratio, further experimental testing is required to investigate the effects of $h_{eff}/L$ ratios that are higher and lower than those analyzed in this study.
3. **Applied precompression level:** The test on specimen W1 demonstrated that applied precompression might prevent the development of high tensile strains in the wall end zones. Even a relatively low level of precompression stress of $0.08f'_{m}$ was found to preclude lateral instability in specimen W1 by reducing its displacement ductility capacity and limiting the magnitude of tensile strains over the wall end zones.

4. **Strain Gradient:** Most of the specimens in the current experimental program were subjected to high axial strains over their plastic hinge regions. These strains were higher than the limits beyond which lateral instability should theoretically take place based on previous experimental studies ([23], [34], [35]): approximately 0.010 and 0.019 over the plastic hinge height for 140 mm and 190 mm thick walls, respectively. It was also observed that strain gradient effects along the wall length appeared to have caused a decrease in out-of-plane displacements before the initiation of crack closure. Evidence of the strain gradient effect was also observed in recent studies on the stability of slender RCSWs ($h_0/t_w$ ratio of 25) by Almeida et al. [30]. In that study, a specimen with a single layer of vertical reinforcement was able to accommodate out-of-plane displacements in excess of the critical displacement, equal to one-half the wall thickness, without succumbing to lateral instability. These observations suggest that the portions of the wall adjacent to an end zone may provide significant out-of-plane stiffness to that end zone, which may help prevent the development of lateral instability. This causes a tendency for face shell spalling and toe-crushing to occur before the development of Class C out of-plane instability.

5. **Vertical reinforcement ratio and bar diameter:** The results of the test on specimen W2, and other full-scale DSRMSW specimens [29] showed that the vertical strain gradient of a wall end zone might cause face shell spalling and/or toe crushing to occur prior to the
development of out-of-plane displacements large enough to precipitate a Class C instability. However, the vertical reinforcing bars become exposed, forcing them to transfer the applied forces across the crushed and spalled regions of the wall. Therefore, it is believed that the vertical reinforcement ratio, or bar diameter in the case of DSRMSWs, is an essential factor affecting the Class B instability. Exposed vertical reinforcing bars in the crushed toe regions may buckle and impose out-of-plane forces to the upper portions of the wall. The resistance against these forces is primarily provided by the out-of-plane stiffness from the portions of the wall adjacent to the end zones experiencing the out-of-plane displacements. This stiffness is dependent principally on the wall thickness and length. If the vertical reinforcing bar diameter is small relative to the wall thickness, the out-of-plane forces produced by the bars deforming in compression may be too small to cause significant out-of-plane displacements; this eventually results in local buckling of the bar, that is, a Class A instability. However, it may be postulated that when the bar diameter is large relative to the wall thickness (say 20M or 25M bars in 140 mm or 190 mm-thick block walls), the forces arising from bar deformation may potentially be high enough to produce out-of-plane displacements sufficient to precipitate instability. An analogy to this scenario could be a two-dimensional problem of a vertically-oriented rigid bar supported by a vertical roller connection at its top and by a rigid connection to a vertically-oriented wire of finite length at its base, with a small eccentricity between the wire axis and the bar axis. When a downward load is applied at the roller, the wire will tend to buckle and impose out-of-plane forces at the bottom of the bar. For the bar to remain vertical, a lateral stiffness must be provided, either in the form of a rotational spring at the roller connection or a set of lateral springs distributed over its height or both. If the moment of inertia of the wire is
minute compared to the spring stiffnesses, the out-of-plane displacements would be small, and the bar may remain essentially vertical during the load application. If the moment of inertia of the wire is large relative to the spring stiffnesses, the out-of-plane displacements could be significant, possibly culminating in instability.

Longitudinal reinforcement ratio and bar diameter may also influence the size of open flexural cracks. A larger longitudinal reinforcement ratio in a DSRMSW with a given $h_w/L_w$ ratio and level of axial precompression, and which has been loaded beyond yield in several successive cycles, suggests that flexural cracks will remain open at higher drifts following load reversal due to a higher axial stiffness of the larger diameter bars ([60]-[62]). This suggests that DSRMSWs with higher reinforcement ratios will be more vulnerable to lateral instability than walls with lower reinforcement ratios. This may help explain the observation that there is a significantly larger body of evidence related to lateral instability of RCSWs than DSRMSWs; the latter walls generally have substantially lower reinforcement ratios and smaller bar sizes (diameters). Therefore, reinforcement ratio and bar size should be considered essential parameters for the out-of-plane stability of DSRMSWs.

6. Eccentricity of vertical reinforcement: In reinforced block masonry construction practice, it is expected that vertical reinforcement will be placed at a small out-of-plane eccentricity with regards to the wall centerline. This was observed in specimens W7 and W8, in which eccentricities of the vertical reinforcing bars were inadvertently introduced during specimen construction and had values of approximately 10 to 20% of the wall thickness. Eccentric bar placements cause early onset of crack closure, as illustrated in Figure 59. With a given crack width, $\Delta_c$, and a given end zone compression force, $C$, the
ratio between the out-of-plane rotations for the zero-eccentricity case (Figure 59a) and the eccentric case (Figure 59b) simply becomes \( t / (t + 2\beta) \), where \( t \) and \( \beta \) are the wall thickness and bar eccentricity, respectively. As out-of-plane rotations decrease, so do the out-of-plane displacements [23]. This results in a lower likelihood of lateral instability occurring before toe-crushing or face shell spalling, i.e. a Class C instability. Similarly, the added moment \( C\beta \) at the top of the bar creates a higher propensity toward bar buckling following toe-crushing and face shell spalling, thus increasing the likelihood of Class A instability and decreasing the probability of Class B instability.

![Figure 59. Effects of eccentric vertical reinforcing bar placement on out-of-plane rotations: zero-eccentricity case a) and a \( \beta \) eccentricity case b).](image)

7. **Boundary elements:** It was observed that for the T-shaped DSRMSW specimens, the flanged boundary elements significantly enhanced the maximum in-plane strength for loading toward the web end zone, but provided negligible strength increase for loading toward the flanged end zone. This is because the lateral load capacity of the specimens was tension-controlled by design to ensure a ductile flexural failure mechanism. A tension-controlled design in this context results in the DSRMSW’s maximum load capacity being directly proportional to the amount of end zone vertical reinforcement until the balanced
reinforcing ratio limit is reached, after which the wall behaviour changes from tension-controlled to compression-controlled. Compression-controlled behaviour is characterized by smaller plastic strains in the vertical reinforcement, decreased ductility, and more rapid strength degradation. This can be illustrated through a comparison of the lateral load-displacement hystereses for the rectangular specimen W8 and the T-shaped specimen W7, for loading toward the web end zone (Figure 54c and d, respectively).

Since the T-shaped specimens, W6 and W7, were tension-controlled by design, the increased area of the compression zone did not significantly increase the maximum strength of the wall because the behaviour was governed by the amount of vertical reinforcement in the web end zone. As a result, the wall’s flexural capacity was only modestly increased by lengthening the moment lever arm for the internal forces. However, an extended moment lever arm caused an increase in the tensile strain demand on the web end zone vertical reinforcement compared to a rectangular wall end zone with the same curvature demand, as illustrated in Figure 60. This indicates a higher vulnerability toward lateral instability for the web end zone of a T-shaped wall based on the close correlation between increasing tensile strains and out-of-plane displacements ([23], [59], [34], [35]).
Out-of-plane displacements in DSRMSWs have been observed to increase most rapidly when the lateral load on the wall is lowest [65]. This is illustrated in Figure 61, which depicts the experimental out-of-plane displacements versus lateral load for specimen W7. Near the point of minimum lateral load, the out-of-plane plate stiffness of the wall is also near its minimum because of the presence of open flexural cracks at both wall end zones. Following a load reversal, the former compression end zone of the wall goes into tension, and cracking is initiated. At the same time, the compression stresses in the former tension zone are low enough to be sustained primarily through the vertical reinforcement spanning the crack, leaving the crack open. As the lateral load increases, the out-of-plane stiffness of the wall increases for two reasons: i) due to axial tension acting to stiffen the new tension end zone, and ii) crack closure acting to stiffen the new compression end zone.

Considering the importance of out-of-plane wall plate stiffness on the development of out-of-plane displacements, uncracked or modestly-cracked flanged end zones are expected to possess a higher resistance toward out-of-plane displacements by increasing the wall out-of-plane plate stiffness. However, once the flanged end zone has become
significantly cracked during reversed-cyclic loading, this increased out-of-plane plate stiffness provided by the flanged end zone will diminish. This is supported by comparing the out-of-plane displacements of the web end zones of the asymmetrically-loaded T-shaped specimen W6 with the T-shaped specimen W7 with the symmetric loading protocol (see Figure 58b and c, respectively). The magnitude of web end zone out-of-plane displacements of these two specimens was very similar at low ductility demands. However, because the flexural cracking at the flanged end zone of specimen W6 remained minor due to the asymmetric loading protocol, the out-of-plane stiffness provided by the flanged end zone remained high throughout the test, and only small increases in the out-of-plane displacement of the web end zone were observed at higher ductility demands. Conversely, as flexural cracking became more significant at the flanged end zone of specimen W7, the out-of-plane stiffness decreased with increasing ductility demands, leading to rising levels of out-of-plane displacements. At drift demands before toe-crushing, the out-of-plane displacements of the web end zone of specimen W7 increased proportionately to the increase in the average axial tensile strain. However, once toe-crushing was initiated in the specimen, the rate of growth in out-of-plane displacements significantly dropped due to a tendency of the relatively small exposed vertical reinforcement beginning to buckle in a Class A instability, rather than producing out-of-plane displacements consistent with a Class B instability.
8. **Symmetry of applied loading protocol:** A comparison of specimens W6 and W7 indicates that an asymmetrical loading protocol, which subjects the flanged end zone to primarily compressive stresses and the web end zone primarily to tensile stresses, may reduce the vulnerability of the web end zone to lateral instability. As discussed in point 1, out-of-plane displacements typically occur most readily when flexural cracks are open over the full wall length. Therefore, because an asymmetrical loading protocol will generally not produce this type of open flexural crack over the entire wall length due to one end zone.
remaining primarily in compression and the other in tension, the likelihood of lateral
instability is reduced in walls subjected to asymmetrical loading protocol.

Similarly, the ratcheting-like behaviour that occurs at the tension end zone during
asymmetric loading due to the flexural cracks widening during a tension excursion and
then failing to completely close following a compression excursion will also result in
decreased out-of-plane rotations. Reversed-cyclic loading with high levels of tension and
even modest in-plane sliding displacements will often produce small dislocated masonry
particles that become wedged along misfit crack interfaces [23]. Due to the low levels of
compression demand at the tension end zone, equilibrium is maintained across the crack
through a combination of compression yielding of the vertical reinforcement and only
partial crushing of the dislocated masonry particles. As a result, the actual clear crack width
that remains open without obstruction is smaller than an alternative scenario where the
dislocated masonry particles would be entirely crushed. Consequently, the effective crack
closure occurs earlier during the loading cycle, thereby decreasing the potential for out-of-
plane rotations and displacements.

The asymmetric loading protocol also caused lengthening of the tension end zone,
e.g. the web end zone of specimen W6, as shown in Figure 62. When the cracks consistently
fail to completely close, the axial strains remain in tension and significantly accumulate
during the subsequent cycles. These open cracks were also found to lead to more extensive
sliding along sliding planes near the base of the wall due to reduced normal stresses and
less effective aggregate interlock.
Figure 62. Averaged axial tension at the web end zone of specimen W6.

9. **Alternative failure mechanisms precluding the development of lateral instability:** The drift demand levels corresponding to face shell spalling, toe-crushing and sliding of DSRMSWs are essential considerations when assessing the potential for lateral instability. DSRMSWs exhibiting ductile flexural behaviour characteristically experience failure mechanisms such as diagonal tension, face shell spalling, toe-crushing, and sliding at high ductility levels [16], which may govern the response of a DSRMSW and preclude the development of lateral instability failure mechanisms. By definition, Class C lateral instability is prevented from occurring if face shell spalling and toe-crushing take place.
However, Class A instability mechanisms frequently occur at the wall toes when the vertical reinforcing bars become exposed due to face shell spalling and toe-crushing, but this instability is localized and unlikely to cause the global failure of the DSRMSW. The occurrence of Class B instability is rare and is influenced by face shell spalling, toe-crushing, sliding, and diagonal tension failure mechanisms. Depending on the vertical reinforcement ratio and bar size, as well as the extent of face shell spalling and toe-crushing, a Class B instability is possible after face shell spalling and toe-crushing take place; as was the case for specimens W2 and W7. However, this instability did not occur in any of the other specimens, which all experienced some degree of face shell spalling and toe-crushing.

Sliding displacements were found to have a significant effect on both the in-plane and out-of-plane response of slender DSRMSWs; as was observed in specimens W3, W4, and W6. With regards to the out-of-plane wall response, sliding displacements produce in-plane rotations of the vertical reinforcing bars across flexural cracks through dowel action, as shown in Figure 63. In-plane rotations act to limit or eliminate out-of-plane rotations along cracks containing the sliding planes because as the cracks close, the bars tend to deform in-plane due to the eccentricity, $\beta$, introduced by the sliding displacements. The highest sliding displacements usually occur near the base of the wall where the flexural cracks are widest, and hence the resistance to sliding is lowest. These flexural cracks near the base of the wall are also crucial for large out-of-plane displacements as they are typically the widest and thus produce the most substantial out-of-plane rotations. Therefore, significant sliding displacements are expected to cause a decrease in the out-of-
plane rotations necessary to develop both Class B and C out-of-plane instabilities; this statement is supported by the observations from the test on specimen W6.

Figure 63. Dowel action mechanism due to sliding displacements.

Furthermore, the dowel action associated with large sliding displacements increases the net strains in the vertical reinforcing bars crossing the sliding planes through kinking behaviour [60]. When high degrees of sliding displacements mobilize along the sliding plane (usually a flexural crack), local grout or concrete spalling begins to occur adjacent to the kinked vertical reinforcing bar. Localized plastic hinges start to develop near the points of maximum curvature in the bars, where tensile stresses due to global flexural and shear stresses, as well as local flexural stresses due to the kinking sum together, as illustrated in Figure 63. The resulting strains may precipitate bar fracture, thus effectively preventing lateral instability.

Based on these observations, DSRMSWs with special end-confinement of the toe regions, such as those proposed by Hart et al. [64], Shedid et al. [27], and Joyal, M.P. [63],
as a means to increase resistance to spalling and toe-crushing, may be potentially more susceptible to lateral instability. However, the slenderness ratios of the end-confined wall specimens tested to-date have been too low to confirm this. If face shell spalling, toe-crushing, diagonal tension, and sliding failures are prevented, lateral instability will likely occur when the wall is subjected to significant ductility demand and the tensile strains of the end-zone reinforcement reach levels that are necessary for the development of critical out-of-plane displacements. Future testing of end-confined DSRMSWs with high slenderness ratios is recommended to determine their vulnerability toward lateral instability.

4.4 Failure Modes

The DSRMSW specimens were found to experience a wide variety of failure modes (see Figure 5 and Figure 64), such as: 1) ductile flexural (DF) - Section 5.2.1; 2) shear-flexure (SF) – Section 5.2.2; 3) sliding (S) – Section 5.2.3; 4) toe-crushing (TC) – session 5.2.4; 5) rocking (RO) – Section 5.2.5; 6) lateral instability (LI) – Section 5.2.6. In many instances, multiple failure modes may be triggered, as shown in Figure 66 and Figure 67. For example, specimen W6 (refer to Table 6) experienced as many as six different failure modes/behaviour types before termination of the test at a lateral drift of 4.14% (see Table 6, Figure 65, and Figure 67b). In contrast, specimen W1 experienced three failure modes before the collapse of the wall at 0.84% drift. The wall design, detailing, and loading was found to typically dictate which failure modes may be encountered as well as which mode will likely govern the response.
Figure 64. Secondary flexure-related DSRMSW failure modes.
Table 6 summarizes the sequence of observed failure modes for each test, whereas Figure 65 provides a graphical representation of the lateral drift ranges of the failure modes. Note that in Table 6, the failure mode shown with the bold and underlined font is the governing failure mode, which is defined here as the failure mode that corresponded most closely to the ultimate lateral load capacity of the specimen. The drift values shown in the parentheses indicate the recorded drift at the onset of the particular failure mode. Note that Figure 65 visually indicates only the main failure mode acting over each of the lateral drift ranges shown, i.e. all prior behavioural and failure mode types were typically coinciding with the subsequent failure modes the followed. In Figure 65, the top of each specimen line represents the end of the test, which does not necessarily represent the incidence of zero lateral capacity of the specimen since some tests were terminated prior to this event due to limitations of the test set up. This section describes the prominent in-plane flexure-dominant failure modes and their interaction during testing of the specimens illustrated in Table 1.
Table 6. Sequence of failure modes for the test specimens.

<table>
<thead>
<tr>
<th>ID</th>
<th>Initial</th>
<th>Secondary</th>
<th>Tertiary</th>
<th>Quarternary</th>
<th>Quinary</th>
<th>Senary</th>
<th>Drift at End of Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1</td>
<td>DF (0.23%) → SF (0.29%) → <strong>TC (0.38%)</strong> - - -</td>
<td>0.84%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W2</td>
<td>DF (0.29%) → <strong>TC (0.49%)</strong> → SL (1.01%) → LI (1.41%) - -</td>
<td>2.56%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W3</td>
<td>DF (0.19%) → <strong>TC (0.59%)</strong> → SL (0.67%) → SF (0.74%) → RO (1.43%) → BF (1.71%)</td>
<td>3.96%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W4</td>
<td>DF (0.14%) → <strong>SL (0.15%)</strong> → RO (0.62%) → BF (1.27%) - -</td>
<td>2.15%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W5</td>
<td>DF (0.18%) → <strong>TC (0.21%)</strong> → SF (0.55%) → RO (0.89%) - -</td>
<td>1.88%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W6</td>
<td>DF (0.15%) → <strong>TC (1.13%)</strong> → LI (1.41%) → SL (2.15%) → RO (3.24%) → BF (3.33%)</td>
<td>4.05%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W7</td>
<td>DF (0.15%) → <strong>TC (1.22%)</strong> → LI (1.75%) → SL (2.01%) → BF (2.96%) -</td>
<td>3.28%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W8</td>
<td>DF (0.15%) → <strong>TC (1.21%)</strong> → BF (2.03%) → SL (2.08%) → RO (2.45%) -</td>
<td>3.25%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Failure Mode Definitions**
- **BF** Bar-Fracture
- **DF** Ductile Flexure
- **LI** Lateral Instability
- **RO** Rocking/Bond-Slip
- **SF** Shear-Flexure
- **SL** Sliding
- **TC** Toe-Crushing
Figure 65. Failure mode lateral drift ranges for the test specimens (refer to Table 6).
Figure 66. Sequence of failure modes affecting specimens: a) W1, b) W2, c) W3, and d) W4.
Figure 67. Sequence of failure modes affecting specimens: a) W5, b) W6, c) W7, and d) W8.
4.5 Summary and Conclusions

Overall, most of the specimens exhibited excellent performance in terms of ductility and energy dissipation in their in-plane response to the simulated in-plane lateral seismic loading, despite demonstrating a variety of different failure modes. Using the EPP ductility approach proposed by Tomazevic [58], the mean value of $\mu_u$ for the DSRMSWs tested in this experimental study were found to be considerably higher than those implied by the CSA S304-14 [17] and NBCC 2015 [18] ductility-related force modification factors for DSRMSWs ($R_d = 3.0$ versus $R_{\text{mean}} = 8.7$). In fact, the 10th percentile of the $\mu_u$ values obtained from the DSRMSW specimens tested approximately corresponded to the $R_d$ value of 3.0 for DSRMSWs as prescribed by CSA S304-14 [17] and NBCC 2015 [18], which may, however, be considered a reasonable percentile derivation, and hence these code $R_d$ values are quite sensible based on the test specimens from this experimental program.

Based on the backbone curves defined in this chapter, the relationships between $Q_y$ and the lateral load at most of the other backbone points ($Q_c$, $Q_u$, and $Q_d$) were reasonably strong, with $Q_c/Q_y \sim 0.66$, $Q_u/Q_y \sim 1.31$, and $Q_d/Q_y \sim 1.05$. Relationships between the displacements at the different backbone points were, however, found to be significantly weaker, yet the mean lateral drift at first-yield was determined to agree well with past experimental findings [12]. The variance in the displacement relations may largely be attributed to the different sequences of failure modes affecting each of the specimens.

Based on this limited number of full-scale DSRMSW specimen tests, it appears that lateral instability is not often encountered for a variety of reasons ranging from high levels of axial precompression to in-plane resistance to out-of-plane displacements due to strain gradients along the wall, to sliding displacements disturbing vertical reinforcing bar deformations. Moreover, the occurrence of lateral instability before the ultimate capacity of the wall being reached was found
to be unlikely for typical DSRMSWs as toe-crushing is generally expected to occur prior to the vertical strains at the tension zone of these walls being high enough to precipitate lateral instability following load reversal. This latter point is essential since DSRMSWs are currently typically designed to achieve their maximum lateral load capacity for a flexure-dominant failure mechanism, which is generally reached at the onset of face shell spalling/splitting, i.e. the initiation of toe-crushing, due to the loss of compression capacity at the wall end zones. As most current masonry design provisions are force-based, this implies that Class B instabilities may not generally be of concern.

The next chapter uses the experimental findings to describe the most prominent failure modes affecting flexure-dominant DSRMSWs and compares the associated provisions from a set of leading masonry design codes to help determine which type of code provisions are most appropriate for each DSRMSW failure mode.
Chapter 5: Failure Modes and Design Code Predictions

5.1 Introduction
This chapter first presents an overview of the most prominent in-plane flexure-related failure modes affecting DSRMSWs, and their relation to one another based on the findings from the experimental phase of this study. Subsequently, the seismic code provisions for DSRMSWs from the masonry design codes of Canada (CSA S304-14 [17]), USA (TMS 402/602-16 [67]), New Zealand (NZS 4230:2004 [68]), and EU (Eurocodes 6 and 8 – BS EN 1996-1-1:2005/BS EN 1998-1:2004 [69], [70]) are first summarized and then compared by analyzing their predicted nominal ULS lateral load capacities relative to the experimental values obtained from the experimental.

5.2 Failure Mode Description Summaries

5.2.1 Ductile-Flexure
Flexural behaviour of DSRMSWs is generally classified as ductile when its capacity is governed by yielding of the vertical reinforcement at the wall ends, and the damage is characterized by a distributed flexural cracking pattern. Accordingly, Table 6 and Figure 65 indicate the onset of ductile flexure (DF) as the point in the test at which the first yield of the vertical reinforcement occurs (e.g. in Figure 65, DF is shown as the failure mode first occurring at the lowest lateral drift on the vertical axis). Adequately reinforced and appropriately detailed DSRMSWs are expected to experience ductile flexure behaviour, which is characterized by high energy dissipation and controlled strength degradation, as confirmed through experimental studies ([27], [71], [71]-[75]). Ductile flexural behaviour is common for DSRMSWs with $h_0/L_w$ ratios greater than 1.5 and those
subjected to relatively low axial compressive stresses, usually less than 10% of $f'_m$. For instance, all of the test specimens from the experimental study experienced ductile flexural behaviour initially [36]. However, at higher axial stress loads, the toe-crushing failure mode is more likely to govern the load capacity of the wall. Flexural behaviour is initiated when horizontal cracks develop at the tension toe of the wall, which is followed by yielding of the vertical reinforcing bars within the wall end zones at lateral drifts that are typically around 0.20% based on the Table 1 test specimens, as shown in Table 6. As strain-hardening begins to develop in the vertical reinforcing bars crossing the horizontal cracks, tensile forces in the reinforcement start to exceed the tensile force capacity in the surrounding masonry at the mortar bed joints. As the lateral displacement demands on the wall continue to increase, new horizontal cracks will begin to form at the bed joints at higher locations up the tension end zone. Figure 68 shows specimens W6 and W8, which both exhibited major horizontal cracking developing at the base of the wall within the plastic hinge height (the region of the wall with the highest curvature demands and plastic strains in the vertical reinforcement), while smaller cracks were observed in the upper portion of the wall (above the plastic hinge region).
Figure 68. Ductile flexural behaviour of DSRMSWs: (a) rectangular specimen W8 at 0.55% drift; and (b) T-shaped specimen W6 at 0.92% drift.

Note that high curvature demands across the plastic hinge region of a DSRMSW may cause the development of high compressive strains in the masonry at the compression toe. A progressive increase in compressive strain eventually causes splitting and spalling of the masonry face shells, which is usually followed by the crushing of the grouted cores. Buckling of the vertical reinforcing bars may then take place within the portion of the wall length that had previously experienced crushing; which can be attributed to the loss of lateral restraint provided by the masonry and grout material.

5.2.2 Shear-Flexure

DSRMSWs with $h_u/L_w$ ratios in the range from 1.0 to 2.0 may be subjected to significant shear stresses under cyclic loads. These stresses cause the development of diagonal cracks along the
direction perpendicular to the principal tensile stress in the wall after the masonry tensile strength has been exceeded. These diagonal cracks often form a stepped pattern along the mortar joints in the central portion of the wall, however alternatively, the cracks may also propagate through the blocks in an X-shaped cracking pattern when the walls are subjected to load reversals. Shear strains and deformations constitute a significant fraction of the total lateral displacement in the walls that experience the combined shear and flexural failure mode [16].

Design and detailing provisions for DSRMSWs generally result in a well-controlled size of diagonal cracks only after ductile flexure has initiated, meaning that shear-flexure will often begin at lateral drifts of 0.25% to 0.75%, as was the case for specimens W1, W3, and W5 (see Table 6). When a wall is lightly reinforced with horizontal bars, a brittle shear failure may develop, which is characterized by a single wide diagonal crack, as shown in Figure 69a. One of the design objectives for the shear design of DSRMSWs is to have a well-distributed diagonal cracking pattern and avoid a shear failure, as shown in Figure 69b [36], [76]-[88]. Once the yielding of the vertical reinforcing bars takes place at the wall end zones, ductile flexural behaviour generally starts to dominate the response and diagonal shear cracks no longer continue to propagate. This behaviour was observed in wall specimens W3 and W5, which are shown in Figure 70.
Figure 69. Effect of horizontal reinforcement on shear behaviour of DSRMSWs: (a) a lightly reinforced wall showing a brittle shear behaviour characterized by a single shear crack; and (b) a heavily reinforced wall showing a ductile shear failure with a distributed cracking pattern.

(a)Brittle shear  
(b) Ductile shear

Figure 70. Distributed cracking pattern in specimens with combined shear and flexural behaviour: (a) specimen W3 at 0.74% drift; and (b) specimen W5 at 0.55% drift.
5.2.3 Sliding

Sliding behaviour typically occurs at the base of low-rise masonry buildings or upper stories of medium-rise masonry buildings [60]. Sliding tends to occur in DSRMSWs with light vertical reinforcement that are subjected to high seismic shear demands and relatively low levels of axial compressive stress. Shear walls with $h_u/L_w$ ratios less than 1.5 (including squat walls) are particularly susceptible to sliding failure [60]-[62]. Sliding behaviour often initiates at higher levels of inelastic flexure deformation demands corresponding to ductility levels above 0.75% drift, as shown in Table 6 for specimens W2, W4, W6, W7, and W8 ([81], [36], [3]). It is, however, important to note when observing the data that specimens W7 and W8 had sliding restraints at their base to help prevent excessive sliding in an attempt to produce a lateral instability failure mode. As a result, the lateral drifts at the onset of sliding for these specimens are significantly larger than the other specimens, as indicated in Table 6 and Figure 65.

Sliding failure may occur in two primary scenarios: when the initial sliding resistance along a sliding plane is exceeded before flexural yielding, or when the sliding resistance is only exceeded after the wall has sustained multiple cycles of inelastic flexural deformations, as previously described [61]. The latter scenario has been observed in many experimental studies since the loading protocols are commonly reversed-cyclic with a high number of incrementally-increasing displacement demands. The sliding resistance decreases with successive cycles of lateral loading due to the combined effects of surface roughness loss along the sliding interface and the development of high inelastic tensile strains in the vertical reinforcement, which cause the sliding plane cracks to take longer to close and the sliding stiffness to soften ([80], [61], [62], [89], [10]).
Laterally-loaded DSRMSWs that are designed to produce a ductile flexural failure mode will typically experience an initial wide sliding crack at the wall base, where the overturning moments are highest. Yielding and high inelastic tensile strains occur in the vertical reinforcement crossing the sliding crack as a result of the shear friction response in the wall’s tension zone, as shown in Figure 71a. At that stage, the wall’s compression toe may provide sufficient shear resistance to prevent further sliding deformations during the current loading cycle. However, upon the load reversal, vertical tension strains will develop at the former compression toe and cause horizontal flexural cracks. Initially, at low lateral load levels, an open crack may span across the entire wall length and force the vertical reinforcement at the compression toe (which was subjected to tension in the previous loading cycle) to resist the full compressive load, as illustrated in Figure 71b. Sliding resistance provided by vertical reinforcement through the shear friction mechanism along the cracked interface will then be dramatically reduced. Most of the sliding resistance must, therefore, be provided by means of dowel action of the vertical reinforcement, which results in a significant drop in lateral sliding stiffness ([10], [89], [90]). A considerable fraction of sliding displacements for a specific loading cycle will occur during this stage. Once the crack at the wall base begins to close within the new compression toe, the sliding resistance increases rapidly as aggregate interlock action and “Coulomb-like” friction activate along with the cracked interface. It can be observed that crushed masonry particles also begin to roll over one another and cause further elongation of the vertical reinforcement, thus producing clamping forces across the crack. When the wall is loaded near its ultimate lateral load capacity, the vertical reinforcing bars at the new compression toe will have yielded in compression, thus providing only modest resistance in the form shear friction compared to the bars at the tension end of the wall that are producing the clamping forces as described earlier [36].
Figure 72 shows the wall displacements recorded during the testing of the DSRMSW specimen W8, which experienced sliding behaviour. It can be seen from the figure that the sliding displacements initially constituted an increasing fraction of the overall wall displacement; however, this contribution converged to approximately one-third of the total displacement as the drift demand exceeded 2.0%. Furthermore, the shear resistance provided by dowel action was found to reach a maximum when sliding deformations caused the vertical bars to deform into diagonal tension ties spanning the two faces of the sliding plane crack. Sliding deformations are expected to stabilize when the sliding stiffness of the wall increases beyond the flexural stiffness, as shown in Figure 71c.

Figure 71. Sliding behaviour initiated as a result of a flexural response: (a) the onset of flexural yielding; (b) the onset of sliding displacements following a load reversal; and (c) sliding resistance arising from dowel action until the crack closure (adapted from Paulay et al. 1982 [10]).

Depending on the magnitude of the sliding resistance, lateral displacements may concentrate along a single sliding plane, thereby precluding damage in other regions of the wall. Some vertical reinforcing bars spanning across the sliding plane also have the potential to fracture due to the combined effects of tensile strain caused by global wall bending, local flexural strain from dowel action, and shear strain from sliding ([3], [67]).
Figure 73 illustrates the sliding-induced cracking pattern for the DSRMSW specimen W3. As evidenced by the behaviour of specimen W3, sliding planes may occur along multiple horizontal sections of a wall (see Figure 73), however there is typically always at least a sliding plane at the wall base, which coincides with a construction joint with a generally smoother interface compared to a sliding plane passing through cracked monolithically-poured grouted cores. Note that sliding planes above the wall base may also occur in between layers of widely-spaced horizontal reinforcing bars due to the discontinuity in shear resistance.

Figure 72. Sliding displacements versus the total wall displacement during the testing of wall specimen W8.
Figure 73. Sliding displacements and multiple sliding planes in specimen: (a) sliding plane (crack) at the wall base at 1.29% drift; and (b) sliding plane at the bottom of the first two masonry courses at 1.66% drift.

5.2.4 Toe-Crushing

A toe-crushing failure mode may occur in DSRMSWs under reversed-cyclic loading due to high inelastic flexural compression deformation demands causing crushing and spalling of the block face shells within the wall’s compression toe. This failure mode often governs the ultimate lateral load capacity of DSRMSWs that initially exhibit ductile flexure behaviour but are subsequently loaded to ductility demands beyond yielding of the vertical reinforcement (walls exceeding 0.50% lateral drift with vertical reinforcement strains as high as 0.03 based on the test data from the specimens). Fundamentally, toe-crushing initiates when the compressive strain of the masonry at the compression toe is exceeded (at compressive strain levels exceeding approximately 0.003 to 0.004 based on the specimen test data). Therefore, if the vertical reinforcement ratio of the wall is high enough to preclude yielding of the vertical reinforcement before toe-crushing (an over-
reinforced condition), toe-crushing may also eliminate the occurrence of ductile flexural behaviour. Since toe-crushing will typically coincide with the incidence of the ultimate lateral load capacity, its appearance is also strongly related to the code-predicted lateral displacement capacity of walls of different ductility levels.

It can be seen from Table 6 that the load capacities of all specimens, except W4, were governed by toe-crushing, and in all of these specimens ductile flexure occurred prior to toe-crushing due to the ductile design of the specimens. For example, specimen W1 first experienced ductile flexural behaviour, which then transitioned into a toe-crushing failure that initiated with vertical splitting cracks developing at the extreme fibre of the compression toe, as shown in Figure 74a. Subsequently, face shell spalling took place, and the wall lost a portion of its effective net cross-section at its base, as shown in Figure 74b. Although some toe-crushing may be expected as a part of the ductile flexural failure, toe-crushing is generally considered as a brittle (non-ductile) failure mode. In the case of significant toe-crushing, the neutral axis of the wall’s cross-section may be forced to shift significantly toward its mid-length (from its original position located closer to the compression toe in a typical DSRMSW); this results in an effective decrease of its flexural capacity. This behaviour was observed during the testing of DSRMSW specimens W3 and W5 (see Figure 75). When studying Table 6 and Figure 65, it is essential to note that the lateral drifts at the onset of toe-crushing for specimens W6-W8 are most likely higher than the other specimens due to the sliding restraint devices implemented for these specimens.

Toe-crushing, accompanied by face shell spalling, may also lead to a reduction in the global lateral stiffness of the wall as the vertical reinforcement in the toe regions becomes exposed. This results in a decrease in the wall’s axial stiffness and the reinforcing bars subjected to compression become susceptible to local buckling (defined as Class A lateral instability in previous chapters of
this thesis), as shown in Figure 76a. Vertical reinforcing bars subjected to numerous cycles of buckling in compression and straightening in tension in the post-yield phase may now begin to experience low-cycle fatigue and ultimately fracture at higher displacement demands (exceeding 1.20%), as shown in Figure 76b. As noted earlier, large sliding displacements will exacerbate the occurrence of bar-fracture as the additional strains due to shear and dowel action decrease the remaining vertical tensile strain capacity of the reinforcing bars [17]. This was observed during the testing of specimens W4 and W6. It can be seen for Table 6 that bar-fracture occurred in both specimens as the final failure mode.

Figure 74. Flexural toe-crushing of specimen W1: (a) the initiation of vertical splitting cracks at 0.55% drift; and (b) the onset of face shell spalling at 0.74% drift.
5.2.5 Rocking

Rocking behaviour is also associated with significant flexural deformations in DSRMSWs, and it may occur after the occurrence of a lap-splice failure, or alternatively in walls with light vertical
reinforcement ([36], [78]). In the first scenario, a lap-splice failure initiated when vertical reinforcing bars projecting from the foundations (dowels) have insufficient lap-splice length. Vertical cracks initially develop at the location of the lap splices, which are subsequently followed by the development of mostly horizontal flexural cracks as well as spalling of the face shells due to tensile stresses over the lap-splice length. Rapid strength degradation ensues as the effectiveness of the tensile stress transfer through the vertical reinforcement is lost, and the wall deformations begin to be governed by rocking action. Note that none of the test specimens examined in the experimental study experienced this type of lap splice rocking behaviour/failure.

Rocking failure may also occur in walls with light vertical reinforcement (low vertical reinforcement ratio) such as specimen W4 (refer to Table 6), particularly within the end zones at even relatively low lateral drift demands (approximately 0.2% lateral drift for specimen W4), or in walls with typical reinforcement but that have experienced extreme ductility demands (typically exceeding 2.50% lateral drift), such as specimen W8. For specimen W4, tension forces generated by the yielded reinforcing bars failed to redistribute following the first flexural crack developing at the wall base, resulting in a lack of new significant secondary flexural cracks in the masonry above, as illustrated in Figure 77a. This behaviour resulted in the wall base crack widening to a very large extent (up to 10 mm prior to the onset of toe-crushing), with only a limited amount of accompanying flexural cracking over the plastic hinge height by the end of the test, i.e. significantly less cracking than would be typical for ductile flexural behaviour. As shown in Figure 78, this sizeable flexural crack at the wall base facilitated rocking action rotations that constituted up to approximately 90% of the total lateral drift at the top of the wall. Note however that the rocking displacement is simply estimated as the curvature produced by the base crack width divided by the length of the wall minus the compression zone depth. As a result, this concentration
of vertical strains in the vertical reinforcement contributed to early bar fracture and a reduction in the predicted inelastic rotational capacity of the wall [91], as shown in Figure 76b. Although this rocking failure mode dissipates a relatively small amount of energy, it may also generate relatively large displacement ductilities [36]. For example, specimen W8 initially experienced rocking behaviour at 2.45% drift following significant toe-crushing, sliding, and two vertical reinforcing bar fractures, but still attained a maximum drift of 3.25% by the end of the test (see Figure 77b).

Figure 77. Rocking failure modes: (a) concentrated damage at the wall base for specimen W4 at 2.21% drift; and (b) rocking following the toe-crushing for specimen W8 at 3.25% drift.
Figure 78. Rocking displacements versus the total wall displacement during the testing of wall specimen W4. Note that only one direction of loading is shown.

5.2.6 Lateral Instability

As discussed in detail in the previous chapter, lateral instability, i.e. out-of-plane buckling of a DSRMSW’s end zone when the wall is subjected to in-plane lateral loading, is a flexural failure mechanism that can theoretically occur as a result of high-cyclic ductility demands (exceeding 1.50% lateral drift) during seismic loading. The most common class of lateral instability is Class A, which also has the least effect on the wall response since it is characterized by the previously-discussed localized buckling of the vertical reinforcing bars following toe-crushing (see Figure 79). Class A instability occurred in most of the specimens that experienced toe-crushing in Table 6. Classes B and C are both global lateral instabilities, and the only difference between the two classes is the prior occurrence of toe-crushing for walls that experience Class B instability (see Figure 79). Note that the failure modes listed as lateral instability (LI) in Table 6 are Class B
instabilities; Class C instability is much rarer and none of the test specimens examined in the experimental study experienced this type of lateral instability, as noted in the previous chapter.

Figure 79. Lateral instability classes in DSRMSWs.

Lateral instability has been observed in a limited number of experimental tests on DSRMSWs; however there is no documented earthquake evidence of its occurrence in real buildings. As mentioned in the literature review, Priestley and He [29] encountered Class B lateral instability during the test on specimen F4-Static 4, which had a hu/tw of 25.6. Shedid et al. [27] also observed significant out-of-plane displacement in specimens Wall 4 and Wall 6, which both had a hu/tw ratio of 18.9. However, the most comprehensive set of test data for Class B lateral instability during the test on specimen W2, which had a hu/tw ratio of 27.1, with additional detail provided in the previous chapter. Specimen W2 is shown in Figure 80a, and its axial strain versus normalized out-of-plane displacement hysteresis is shown again in Figure 81. Large out-of-plane displacements were also recorded during the testing of specimen W7, which had a hu/tw ratio of 20.1; however these displacements did not culminate in lateral instability (see Figure 80b).
Figure 80. Lateral instability: (a) specimen W2 at 2.58% drift; and (b) specimen W7 at 2.95% drift.

Figure 81. Average vertical strain versus normalized out-of-plane displacement hysteresis for specimen W2.
5.3 Design Code Capacity Prediction Comparisons

5.3.1 International Masonry Design Codes

Most masonry design codes contain provisions for determining the lateral load, and displacement capacities of DSRMSWs for the prominent in-plane flexure-dominant failure modes described earlier in the chapter. These provisions may be either explicit (containing quantitative capacity prediction equations) or implicit (providing only empirical restrictions and geometrical calculations). Note that only the unfactored (nominal) capacities from the ULS seismic provisions for DSRMSWs are considered here (as opposed to the Allowable Stress Design (ASD) approach). Provisions specific to fully-grouted hollow concrete block units have been used where applicable since the code predictions have been validated against the results of the experimental test series of eight full-scale fully grouted DSRMSW specimens described earlier in this chapter. Note that the investigated provisions are pertinent to the most ductile DSRMSW class considered by each code.

This section first briefly describes the selected masonry design codes (TMS 402/602-16 [67], CSA S304-14 [17], NZS 4230:2004 [68], and EN 1996-1-1:2005/EN 1998-1:2004 [69], [70]) and then provides summary comparisons of both the explicit and implicit code provisions.

In the USA, TMS 402-16: Building Code Requirements for Masonry Structures provides the current building code requirements for masonry structures, whereas TMS 602-16 provides the specifications for masonry structures [67]. Both were developed by The Masonry Society (TMS). The code and specifications cover the design and construction of masonry structures and include a side-by-side commentary on the details and rationale behind many of the design provisions. The code covers both unreinforced and reinforced masonry structures and provides minimum requirements for both ASD and ULS design methodologies. The code provisions address both the non-seismic and seismic design of masonry structures.
In Canada, *CSA S304-14: Design of Masonry Structures* provides the current design and construction provisions for unreinforced, reinforced, prefabricated, and prestressed masonry [17]. The code was produced by the Canadian Standards Association (CSA) Technical Committee. The code primarily uses the ULS design philosophy and covers all seismic and non-seismic provisions for the design of unreinforced and reinforced masonry structures. Empirical design provisions are also included in an appendix.

In New Zealand, *NZS 4230:2004: Design of Reinforced Concrete Masonry Structures* provides the current design provisions for the design of reinforced and prestressed concrete masonry structures; unreinforced structural masonry is not covered by this code [68]. The code was established under the direction of the Building and Civil Engineering Divisional Committee for the Standards Council of New Zealand. The design philosophy of the code is based on the ULS design approach. Many of the code provisions have been developed with special consideration toward the satisfactory seismic performance of reinforced masonry structures. The code contains provisions that are followed immediately by pertinent commentary information and includes cross-references to *NZS 3101.1:2006: The Design of Concrete Structures* [92], which provides additional guidance for the seismic design of structures.

*Eurocode 6 – EN 1996-1-1: Design of Masonry Structures – Part 1:01: Common Rules for Reinforced and Unreinforced Masonry Structures* provides the current design provisions for European unreinforced, reinforced, prestressed, and confined masonry structures [69]. The code was published by the European Committee for Standardization (CEN) and was designed for use with the National Application Document (NAD) of member countries. This code uses the ULS design philosophy and does not cover seismic-specific design provisions, which are instead included in *Eurocode 8 – EN 1998-1:2004: Design of Structures for Earthquake Resistance* [70].
This code provides seismic provisions for all types of structures but also includes a chapter that specifically pertains to masonry buildings.

### 5.3.2 Comparison of Explicit Force-Based Code Provisions

Table 7 to Table 9 summarize the key explicit force-based code provisions for DSRMSWs for ductile-flexure, shear-flexure, and sliding for each of the codes investigated in this chapter (TMS 402/602-16 [67], CSA S304-14 [17], NZS 4230:2004 [68], and EN 1996-1-1:2005/EN 1998-1:2004 [69], [70]). These tables are followed by Figure 82, Figure 83, and Figure 84, which show a comparison of code-predicted capacities and experimental values from the eight DSRMSW specimens tested in the experimental phase of this study. Note that the affixes -1 and -2 for the specimen labelling refer to the two directions of loading for each specimen (W1 to W8) and that unit material factor values were used for the capacity provisions. The horizontal line on the charts shows an exact normalized prediction \( (V_{\text{predicted}}/V_{\text{experimental}} = 1.0) \), while normalized predictions greater than 1.0 represent overpredictions and normalized predictions less than 1.0 represent underpredictions.
Table 7. Ductile-flexure code provision summary.

<table>
<thead>
<tr>
<th>Code</th>
<th>Requirement</th>
<th>Code Clause</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>TMS 402/602-16</td>
<td>$M_u \leq \varphi M_n$</td>
<td>Cl.9.3.4.1.1</td>
<td>$A_s$ = cross-sectional area of non-prestressed longitudinal tensile reinforcement, mm$^2$</td>
</tr>
<tr>
<td></td>
<td>$\frac{M_u}{V_u d_v} \geq 1$,</td>
<td>Cl.9.3.3.2.1</td>
<td>$A_{reqd}$ = cross-sectional area of flexural tensile reinforcement required to maintain axial equilibrium, mm$^2$</td>
</tr>
<tr>
<td></td>
<td>$A_s \leq A_{reqd}$ and $\varepsilon_s \leq \alpha \varepsilon_y$</td>
<td>Cl.9.3.3.2.3</td>
<td>$c$ = the distance from the maximum compressive fibre to the neutral axis, mm</td>
</tr>
<tr>
<td></td>
<td>For $\frac{M_u}{V_u d_v} \leq 1$,</td>
<td>Cl.9.3.3.2.4</td>
<td>$f_s$ = calculated tensile or compressive stress in reinforcement, MPa</td>
</tr>
<tr>
<td></td>
<td>$f_s = \frac{M_n}{d_v}$ = specified yield strength of steel reinforcement, MPa</td>
<td></td>
<td>$M_n$ = nominal moment strength, N-mm</td>
</tr>
<tr>
<td></td>
<td>$M_u$ = factored moment demand (including second-order effects where applicable), N-mm</td>
<td></td>
<td>$V_u$ = factored shear demand, N</td>
</tr>
<tr>
<td></td>
<td>Plane sections remain plane assumption</td>
<td>Cl.9.3.2(d)</td>
<td>$\alpha$ = 1.5 for ordinary RMSWs; 3.0 for intermediate RMSWs; 4.0 for special RMSWs</td>
</tr>
<tr>
<td></td>
<td>Neglect masonry tensile strength</td>
<td>Cl.9.3.2(f)</td>
<td>$\varepsilon_{mu}$ = maximum usable compressive strain of masonry, mm/mm</td>
</tr>
<tr>
<td></td>
<td>Limiting masonry strain $\varepsilon_{mu} \leq 0.0025$ for concrete block units</td>
<td>Cl.9.3.2(c)</td>
<td>$\varepsilon_s$ = calculated tensile or compressive strain in reinforcement, MPa</td>
</tr>
<tr>
<td></td>
<td>Bilinear stress-strain relation for steel with yield at $f_s = f_y$</td>
<td>Cl.9.3.2(e)</td>
<td>$\varepsilon_y$ = yield strain of steel reinforcement, mm/mm</td>
</tr>
<tr>
<td></td>
<td>Strain compatibility between masonry, grout, and steel</td>
<td>Cl.9.3.2(a)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Masonry compressive stress of $0.8 f_m'$ uniformly distributed over an equivalent compression stress block extending $0.8c$ from the maximum compressive fibre</td>
<td>Cl.9.3.2(g)</td>
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<tr>
<td>Code</td>
<td>Requirement</td>
<td>Code Clause</td>
<td>Remarks</td>
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<tr>
<td>CSA S304-14</td>
<td>$M_f \leq M_r$</td>
<td>Cl.10.2</td>
<td>$A_s = \text{cross-sectional area of vertical reinforcement in tension, mm}^2$</td>
</tr>
<tr>
<td></td>
<td>$\alpha f'_m = 0.85 \varphi_m \chi f'_m$</td>
<td>Cl.10.2.6</td>
<td>$c = \text{the distance from the fibre of maximum compressive strain to the neutral axis, mm}$</td>
</tr>
<tr>
<td></td>
<td>$\beta_1 c = \frac{P_f + \varphi_s f_s A_s}{0.85 \varphi_m \chi f'_m t_w/f}$</td>
<td>Cl.10.2.6</td>
<td>$f'_m = \text{specified masonry compressive strength, MPa}$</td>
</tr>
<tr>
<td></td>
<td>Plane sections remain plane assumption</td>
<td>Cl.10.2.1</td>
<td>$P_f = \text{factored axial load, N}$</td>
</tr>
<tr>
<td></td>
<td>Neglect masonry tensile strength</td>
<td>Cl.10.2.4</td>
<td>$t_f = \text{thickness of wall flange, mm}$</td>
</tr>
<tr>
<td></td>
<td>Masonry stress-strain relation taken as parabolic, trapezoidal, or any other shape that results in the prediction of strength in substantial agreement with results of comprehensive tests</td>
<td>Cl.10.2.5</td>
<td>$t_w = \text{thickness of wall web, mm}$</td>
</tr>
<tr>
<td></td>
<td>Limiting masonry strain $\varepsilon_{mu} \leq 0.003$</td>
<td>Cl.10.2.3</td>
<td>$\alpha = \text{stress of equivalent rectangular masonry stress block}$</td>
</tr>
<tr>
<td></td>
<td>Bilinear stress-strain relation for steel with yield at $f_s = f_y$</td>
<td>Cl.10.2.3</td>
<td>$\beta_1 = 0.8$ for $f'_m \leq 20 \text{ MPa}$ and $0.8 - 0.1$ for each 10 MPa of strength over 20 MPa</td>
</tr>
<tr>
<td></td>
<td>Strain compatibility between masonry, grout, and steel</td>
<td>Cl.16.9.7</td>
<td>$\varphi_m = \text{resistance factor for masonry}$</td>
</tr>
<tr>
<td></td>
<td>$c \leq 0.125l_w$</td>
<td>Cl.10.2.3.1</td>
<td>$\varphi_s = \text{resistance factor for steel}$</td>
</tr>
<tr>
<td></td>
<td>NZS 4230:2004</td>
<td>$\varphi M_n \geq M^*$</td>
<td>$\chi = \text{factor used to account for the direction of compressive stress in a masonry member relative to the direction used for the determination of } f'_m$: 0.5 for compressive forces applied normal to the vertical mortar joints and the grout is not horizontally continuous in the zone of compression; 0.7 for compressive forces applied normal to the vertical mortar joints, and the grout is horizontally continuous in the zone of compression; 1.0 for compressive forces applied normal to the horizontal mortar joints</td>
</tr>
<tr>
<td></td>
<td>Plane sections remain plane assumption</td>
<td>Cl.10.2.2.2</td>
<td>$c = \text{the distance from the fibre of maximum compressive strain to the neutral axis measured in a direction perpendicular to that axis, mm}$</td>
</tr>
</tbody>
</table>

$\varphi_m = \text{maximum usable compressive strain of masonry, mm/mm}$

$\varphi_s = \text{resistance factor for steel}$

$\chi = \text{factor used to account for the direction of compressive stress in a masonry member relative to the direction used for the determination of } f'_m$: 0.5 for compressive forces applied normal to the vertical mortar joints and the grout is not horizontally continuous in the zone of compression; 0.7 for compressive forces applied normal to the vertical mortar joints, and the grout is horizontally continuous in the zone of compression; 1.0 for compressive forces applied normal to the horizontal mortar joints

$\varphi M_n \geq M^*$

Plane sections remain plane assumption
### Ductile-Flexure

<table>
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<tr>
<th>Code</th>
<th>Requirement</th>
<th>Code Clause</th>
<th>Remarks</th>
</tr>
</thead>
</table>
| NZS 4230:2004 | Neglect masonry tensile strength | Cl.10.2.2.5 | \( f_y = \) specified yield strength of steel reinforcement, MPa  
\( \varepsilon_{mu} = \) maximum usable compressive strain of masonry, mm/mm  
\( \phi = \) strength reduction factor (Cl.3.4.7) |
| Masonry compressive stress of \( 0.85f'_m \) uniformly distributed over an equivalent compression stress block extending \( 0.85c \) from the maximum compressive fibre | Cl.10.2.2.6(i) | |
| Limiting masonry strain \( \varepsilon_{mu} \leq 0.003 \) | Cl.10.2.2.3 | |
| Strain compatibility between masonry, grout, and steel | Cl.10.2.2.1 | |
| Bilinear stress-strain relation for steel with yield at \( f_s = f_y \) | Cl.10.2.2.5 | |
| Eurocodes 6 and 8 – BS EN 1996-1-1:2005/BS EN 1998-1:2004 | \( S_d \leq R_d \) | Cl.6.4.2(1) | \( c = \) the distance from the fibre of maximum compressive strain to the neutral axis measured in a direction perpendicular to that axis, mm  
\( f_d = \) design compressive strength of masonry, MPa  
\( f_k = \) characteristic compressive strength of masonry in the direction of loading, MPa  
\( L = \) horizontal wall length, mm  
\( R_d = \) design load resistance of reinforcement masonry member, N-mm  
\( S_d = \) applied load on reinforced masonry member, N-mm  
\( \varepsilon_{mu} = \) maximum usable compressive strain of masonry, mm/mm  
\( \varepsilon_s = \) calculated tensile strain in reinforcement, MPa  
\( \gamma_m = \) partial safety factor for masonry properties |
<p>| Plane sections remain plane assumption | Cl.6.4.1(1) | Eq. 6.29 |
| Neglect masonry tensile strength | Cl.6.4.1(1) | |
| Strain compatibility between masonry, grout, and steel | Cl.6.4.1(1) | |
| Limiting masonry strain ( \varepsilon_{mu} \leq 0.0035 ) | Cl.6.4.2(5) | |</p>
<table>
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<tr>
<th>Code</th>
<th>Requirement</th>
<th>Code Clause</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eurocodes 6 and 8 – EN 1996-1-1:2005/EN 1998-1:2004</td>
<td>Masonry compressive stress of $f_d$ uniformly distributed over an equivalent compression stress block extending $0.8c$ from the maximum compressive fibre</td>
<td>Cl.6.4.2(5)</td>
<td>See the previous page.</td>
</tr>
<tr>
<td></td>
<td>Masonry stress-strain relation taken as parabolic, trapezoidal, or rectangular</td>
<td>Cl.6.4.1(1)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\varepsilon_s \leq 0.01$</td>
<td>Cl.6.4.2(2)</td>
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<tr>
<td></td>
<td>$c \leq 0.4L$</td>
<td>Cl.6.4.2(3)</td>
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</tbody>
</table>
Table 8. Shear-flexure code provision summary.

<table>
<thead>
<tr>
<th>Code</th>
<th>Requirement</th>
<th>Code Clause</th>
<th>Remarks</th>
</tr>
</thead>
</table>
| TMS 402/602-16 | \( V_u \leq \varphi V_n \) | Cl.9.3.4.2.3 | \( A_{nv} = \) net cross-sectional area of the member subjected to considered shear demand, mm
<p>| | ( V_n = V_{nm} + V_{ns} ) | Cl.9.3.4.1.2, Eq. 9-17 | ( A_v = ) cross-sectional area of shear reinforcement within distance, s, mm(^2) ( d_v = ) member depth in the direction of considered shear demand, mm |
| | ( V_n \leq 6A_{nv}\sqrt{f_m'} ) | Cl.9.3.4.1.3, Eq. 9-18 | ( f_m' = ) specified masonry compressive strength, MPa ( f_{yt} = ) specified yield strength of transverse reinforcement, MPa ( M_u = ) factored moment demand (including second-order effects where applicable), N-mm |
| where | ( \frac{M_u}{V_u d_v} \leq 0.25 ) | Cl.9.3.4.1.3, Eq. 9-19 | ( P_u = ) factored axial demand, N ( s = ) reinforcement spacing along member length, mm |
| | ( V_n \leq 4A_{nv}\sqrt{f_m'} ) | Cl.9.3.4.1.3, Eq. 9-20 | ( V_n = )nominal shear strength, N ( V_{nm} = )nominal shear strength provided by masonry, N ( V_{rs} = )nominal shear strength provided by reinforcement, N ( V_u = ) factored shear demand, N ( \varphi = ) strength reduction factor |
| | where ( \frac{M_u}{V_u d_v} \geq 1.0 ) | | |
| | ( V_n = \left[ 4.0 - 1.75 \left( \frac{M_u}{V_u d_v} \right) \right] A_{nv}\sqrt{f_m'} + 0.25P_u + 0.5 \left( \frac{A_v}{s} \right) f_{yt} d_v ) | | |
| CSA S304-14 | ( V_f \leq V_r ) | Cl.10.10 | ( A_v = ) cross-sectional area of shear reinforcement within distance, s, mm(^2) ( b_w = )effective wall web width, mm ( d_v = )member depth in the direction of considered shear demand, mm |
| | ( V_r = \varphi_m(v_m b_w d_v + 0.25P_d) ) | Cl.10.10.1.1 | ( f_m' = )masonry compressive strength, MPa ( f_y = )yield strength of reinforcement, MPa ( M_f = )factored moment demand, N-mm ( P_u = )axial compressive load on the section under consideration, based on 0.9 dead load + factored axial load arising from bending in coupling beams where applicable, N-mm ( s = )reinforcement spacing along member length, mm |</p>
<table>
<thead>
<tr>
<th>Code</th>
<th>Requirement</th>
<th>Code Clause</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>CSA S304-14</td>
<td>$v_m = 0.16 \left( 2 - \frac{M_f}{V_f d_v} \right) \sqrt{f_m'}$</td>
<td>Cl.10.10.4</td>
<td>$v_m = \text{shear strength of masonry, MPa}$</td>
</tr>
<tr>
<td></td>
<td>$V_r = \text{nominal shear strength, N}$</td>
<td></td>
<td>$V_f = \text{factored shear demand, N}$</td>
</tr>
<tr>
<td></td>
<td>$\varphi_m = \text{resistance factor for masonry}$</td>
<td></td>
<td>$\varphi_s = \text{resistance factor for steel}$</td>
</tr>
<tr>
<td>NZS 4230:2004</td>
<td>$\varphi V_n \geq V^*$</td>
<td>Cl.10.3.2.1</td>
<td>$A_{ps} = \text{cross-sectional area of prestressed reinforcement in flexural tension zone, mm}^2$</td>
</tr>
<tr>
<td></td>
<td>$V_n = \varphi v_n b_w d$</td>
<td>Eq. 10-3</td>
<td>$A_s = \text{cross-sectional area of non-prestressed reinforcement, mm}^2$</td>
</tr>
<tr>
<td></td>
<td>$v_n = v_m + v_p + v_s$</td>
<td>Cl.10.3.2.2</td>
<td>$A_v = \text{cross-sectional area of shear-friction reinforcement, mm}^2$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Eq. 10-5</td>
<td>$A_{vmin} = \text{minimum permissible cross-sectional area of shear-friction reinforcement, mm}^2$</td>
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<tr>
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<td>$b_w = \text{effective width of the wall, mm; for flanged walls, } b_w \leq \text{least of (a) half the distance to the next web, (b) 8 times } t_v, (c) 1/9 the height of the shear wall}$</td>
</tr>
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<td></td>
<td>(Cl.3.6.1.7(a)(b))</td>
</tr>
<tr>
<td></td>
<td>$v_m = (C_1 + C_2)v_{bm}$</td>
<td>Cl.10.3.2.6</td>
<td>$C_1 = \text{shear strength coefficient } (\text{Cl.10.3.2.6(a)})$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Eq. 10-6</td>
<td>$C_2 = \text{shear strength coefficient } (\text{Cl.10.3.2.6(c)})$</td>
</tr>
<tr>
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<td>Cl.10.3.2.2</td>
<td>$C_3 = \text{shear strength coefficient } (\text{Cl.10.3.2.11})$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Eq. 10-7</td>
<td>$d = \text{distance from extreme compression fibre to centroid of longitudinal tension}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\text{reinforcement, but need not be less than 0.8 times the horizontal length of the wall in}$</td>
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<tr>
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<td></td>
<td>$\text{the direction of the applied shear, or 0.9 times the overall depth of the member in the}$</td>
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<td></td>
<td></td>
<td>$\text{plan of the loading, mm}$</td>
</tr>
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<td></td>
<td>$h_w = \text{effective wall height in the plane of applied loading, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cl.10.3.2.11</td>
<td>$L_{w} = \text{horizontal wall length, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Eq. 10-8</td>
<td>$N' = \text{design axial load in compression at a given eccentricity, N}$</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>$p_w = \text{vertical reinforcement ratio}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cl.10.3.2.11</td>
<td>$s = \text{spacing of shear reinforcement in a direction parallel to longitudinal}$</td>
</tr>
<tr>
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<td></td>
<td>Eq. 10-6a</td>
<td>$\text{reinforcement, mm}$</td>
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<tr>
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<td></td>
<td></td>
<td>$v_{bm} = \text{basic type dependent shear strength of masonry, MPa}$</td>
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<tr>
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<td></td>
<td></td>
<td>$v_m = \text{shear strength of masonry, MPa}$</td>
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<td></td>
<td>$v_n = \text{shear stress corresponding to } V_n, \text{ MPa}$</td>
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<tr>
<td></td>
<td></td>
<td>Cl.10.3.2.11</td>
<td>$v_p = \text{shear strength of prestressed reinforcement, MPa}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Eq. 10-6b</td>
<td>$v_s = \text{shear strength of non-prestressed reinforcement, MPa}$</td>
</tr>
<tr>
<td>Code</td>
<td>Requirement</td>
<td>Code Clause</td>
<td>Remarks</td>
</tr>
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</tr>
<tr>
<td>NZS 4230:2004</td>
<td>$A_{v_{\text{min}}} = \frac{0.15 b w s}{f_y}$</td>
<td>Cl.10.3.2.11</td>
<td>V* = shear demand of factored loads, N</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Eq. 10-9</td>
<td>$V_n$ = factored shear resistance, N</td>
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<td>$\phi$ = strength reduction factor (Cl.3.4.7)</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>$\phi$ = Angle formed between lines of axial load action and resulting reaction on a component, see Cl. 10.3.2.7, degrees</td>
</tr>
<tr>
<td>Eurocodes 6 and 8 – EN 1996-1-1:2005/EN 1998-1:2004</td>
<td>$V_{sd} \leq V_{\text{rd1}} + V_{\text{rd2}}$</td>
<td>Cl.6.5.2</td>
<td>$A_{sw}$ = area of horizontal shear reinforcement, mm$^2$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Eq. 6.44</td>
<td>$f_{vd}$ = design shear strength of masonry, MPa</td>
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<td>$f_{vk}$ = characteristic shear strength of masonry, MPa</td>
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<td>$f_{vk0}$ = characteristic shear strength of masonry under zero compressive load, MPa</td>
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<td>$f_{yd}$ = design yield strength of shear reinforcement, MPa</td>
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<td></td>
<td>$f_{yk}$ = characteristic yield strength of reinforcing steel, MPa</td>
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<td></td>
<td>$L$ = wall length, mm</td>
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<td></td>
<td></td>
<td></td>
<td>$s$ = vertical spacing of horizontal shear reinforcement, mm</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>$t$ = wall thickness, mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cl.3.6.2</td>
<td>$V_{\text{rd1}}$ = masonry contribution to shear resistance, N</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Eq. 3.3a</td>
<td>$V_{\text{rd2}}$ = shear reinforcement contribution to shear resistance, N</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$V_{sd}$ = design value of the applied shear load, N</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>$\gamma_m$ = partial safety factor for masonry properties (Table 2.4.3)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\gamma_s$ = partial safety factor for steel (Table 2.4.3)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\sigma_d$ = the design compressive stress perpendicular to the shear in the member at the level under consideration, using the appropriate load combination, MPa</td>
</tr>
<tr>
<td></td>
<td>$f_{vk} = f_{vk0} + 0.4 \sigma_d$</td>
<td>Cl.3.6.2</td>
<td></td>
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<tr>
<td></td>
<td>$f_{vd} = \frac{f_{vk}}{\gamma_m}$</td>
<td>Cl.3.6.2</td>
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<tr>
<td></td>
<td>$f_{yd} = \frac{f_{yk}}{\gamma_s}$</td>
<td>Cl.3.6.2</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$V_{\text{rd2}} = \frac{A_{sw}}{s} f_{yd} L$</td>
<td>Cl.6.5.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Eq. 6.45</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\frac{V_{\text{rd1}} + V_{\text{rd2}}}{tL} \leq 2.0 \text{ MPA}$</td>
<td>Cl.6.5.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Eq. 6.46</td>
<td></td>
</tr>
</tbody>
</table>
Table 9. Sliding code provision summary.

<table>
<thead>
<tr>
<th>Code</th>
<th>Requirement</th>
<th>Code Clause</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>TMS 402/602-16</td>
<td>$V_u \leq \varphi V_{nf}$</td>
<td>Cl.9.3.6.5</td>
<td>$A_{sp}$ = cross-sectional area of reinforcement within the net shear area, perpendicular to and crossing the horizontal shear plane, mm$^2$</td>
</tr>
<tr>
<td></td>
<td>$V_{nf} = \mu(A_{sp}/f_y + P_u) \geq 0$</td>
<td>Cl.9.3.6.5 Eq. 9-33</td>
<td>$A_{nc}$ = net cross-sectional area between the neutral axis of bending and the maximum compressive fibre calculated at the nominal moment capacity of the section, mm$^2$</td>
</tr>
<tr>
<td></td>
<td>where $\frac{M_u}{V_u d_v} \leq 0.5$</td>
<td></td>
<td>$d_v$ = member depth in the direction of considered shear demand, mm</td>
</tr>
<tr>
<td></td>
<td>$V_{nf} = 0.42f'<em>m A</em>{nc}$</td>
<td>Cl.9.3.6.5 Eq. 9-34</td>
<td>$M_u$ = factored moment demand (including second-order effects where applicable), N-mm</td>
</tr>
<tr>
<td></td>
<td>where $\frac{M_u}{V_u d_v} \geq 1.0$</td>
<td></td>
<td>$P_u$ = factored axial demand, N</td>
</tr>
<tr>
<td></td>
<td>$V_{nf} = \mu f_y A_v$</td>
<td></td>
<td>$V_{nf}$ = nominal shear-friction strength, N</td>
</tr>
<tr>
<td></td>
<td>$V_u$ = factored shear demand, N</td>
<td></td>
<td>$\varphi = $ strength reduction factor</td>
</tr>
<tr>
<td></td>
<td>$\mu = $ coefficient of friction taken as 1.0 for masonry-to-masonry or masonry-to-roughened-concrete sliding planes and 0.7 for masonry-to-smooth-concrete or masonry-to-bare-steel sliding planes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CSA S304-14</td>
<td>$V_f \leq V_r$</td>
<td>Cl.10.10</td>
<td>$A_v$ = cross-sectional area of shear reinforcement within distance, s, mm$^2$</td>
</tr>
<tr>
<td></td>
<td>$C = \varphi_m \mu C$</td>
<td>Cl.10.10.5.1</td>
<td>$C$ = compressive force in the masonry acting normal to the sliding plane, normally taken as $P_v$ plus the factored tensile force at yield of the vertical reinforcement that is detailed to develop yield strength on both sides of the sliding plane</td>
</tr>
<tr>
<td></td>
<td>$V_f = \varphi_m \mu C$</td>
<td></td>
<td>$P_u$ = axial compressive load on the section under consideration, based on 0.9 dead load + factored axial load arising from bending in coupling beams where applicable, N</td>
</tr>
<tr>
<td></td>
<td>$C = (f'_y A_v + P_u)$</td>
<td></td>
<td>$V_f = $ shear demand of factored loads, N</td>
</tr>
<tr>
<td></td>
<td>$V_r = $ factored shear resistance, N</td>
<td></td>
<td>$\varphi_m = $ resistance factor for masonry</td>
</tr>
<tr>
<td>Code</td>
<td>Requirement</td>
<td>Code Clause</td>
<td>Remarks</td>
</tr>
<tr>
<td>-------------------</td>
<td>----------------------------</td>
<td>--------------------------</td>
<td>-------------------------------------------------------------------------</td>
</tr>
<tr>
<td>NZS 4230:2004</td>
<td>( \varphi V_n \geq V^* )</td>
<td>Cl.10.3.2.1</td>
<td>( \mu = ) coefficient of friction taken as 1.0 for masonry-to-masonry or masonry-to-roughened-concrete sliding planes and 0.7 for masonry-to-smooth-concrete or masonry-to-bare-steel sliding planes</td>
</tr>
<tr>
<td></td>
<td>( V_n = \mu_f (f_y A_{vf} + N^*) )</td>
<td>Eq. 10-3</td>
<td>( A_{vf} = ) cross-sectional area of shear-friction reinforcement, mm(^2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cl.10.3.2.13</td>
<td>( N^* = ) design axial load in compression at a given eccentricity, N</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Eq. 10-10</td>
<td>( V^* = ) shear demand of factored loads, N</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>( V_n = ) factored shear resistance, N</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>( \varphi = ) strength reduction factor (Cl.3.4.7)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>( \mu_f = ) coefficient of friction, taken as 1.0 for monolithic, unfinished but clean surfaces, and 0.70 for all other situations</td>
</tr>
<tr>
<td>Eurocodes 6 and</td>
<td>For floors, roofs, or ring beams bearing on walls, the frictional resistance shall be capable of transferring the lateral loads</td>
<td>Cl.8.5.1.3</td>
<td>Not applicable</td>
</tr>
</tbody>
</table>
5.3.2.1 Ductile-Flexure

As discussed in the previous chapter, it was found that ductile-flexure generally dominated the response of all the specimens during testing, except W4. Figure 82 shows that ductile-flexure failure has typically the lowest normalized capacity predictions: a median (MED) normalized capacity prediction is 0.70, while a mean (MEAN) prediction is 0.73 with a standard deviation (STD) of 0.16 and a coefficient of variation (COV) of 22.0%. As mentioned earlier, it is not surprising that ductile-flexure governed the predicted lateral load capacities considering that the specimens were explicitly designed to fail in a ductile flexural failure mode. Some of the codes, e.g. TMS 402/602-16 [67] and Eurocode 6 [69], provide restrictions on the maximum tensile strain permitted in the vertical reinforcement, size of the compression zone, and the tied vertical compression reinforcement in the calculations (see Table 10). However, the general design approach is the same for the considered codes, and it is primarily based on beam theory. The common assumptions are: i) plane sections remain plane, ii) masonry tensile strength is disregarded in the design, iii) steel has a bilinear stress-strain relation, and iv) there is strain compatibility between the masonry, grout and steel. All predictions were relatively close to one another. Ranked in descending order, the predicted values were highest for CSA S304-14 [17], followed by NZS 4230:2004 [68], TMS 402/602-16 [67], and Eurocode 6 [69] (the latter two codes showed consistently lower capacity predictions due to the imposed tensile strain limits).
Figure 82. Code predictions for the ductile flexural capacity normalized to the respective experimental results.

Table 10. Summary of key flexural design requirements for DSRMSWs from different codes.

<table>
<thead>
<tr>
<th>Code</th>
<th>TMS 402-602</th>
<th>CSA S304-14</th>
<th>NZS 4230:2004</th>
<th>Eurocode 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry Material Resistance Factor</td>
<td>0.90</td>
<td>0.60</td>
<td>0.85</td>
<td>0.60</td>
</tr>
<tr>
<td>Steel Material Resistance Factor</td>
<td>0.90</td>
<td>0.85</td>
<td>0.85</td>
<td>0.85</td>
</tr>
<tr>
<td>Masonry Compression Zone Stress Limit</td>
<td>0.80 $f'_m$</td>
<td>~0.85 $f'_m$</td>
<td>0.85 $f'_m$</td>
<td>$f_d (f'_m)$</td>
</tr>
<tr>
<td>Masonry Compression Strain Limit</td>
<td>0.0025</td>
<td>0.0030</td>
<td>0.0030</td>
<td>0.0035</td>
</tr>
<tr>
<td>Steel Tension Strain Limit</td>
<td>0.008</td>
<td>None</td>
<td>None</td>
<td>0.0100</td>
</tr>
<tr>
<td>Tied Compression Steel Considered in Calculations</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>Compression Zone Length (equivalent rectangular stress block)</td>
<td>0.8 c</td>
<td>~0.8 c</td>
<td>0.85 c</td>
<td>0.8 c</td>
</tr>
</tbody>
</table>
5.3.2.2 Shear-Flexure

In general, all of the codes treat shear-flexure and shear identically and have relatively similar calculation methodologies for determining shear-flexure resistance, e.g. a sum of components attributed to the masonry, effect of horizontal reinforcement, and axial compression stress, where applicable. In general, the provisions comprise a series of equations to determine each of these components, with the CSA S304-14 [17] and Eurocode 6 [69] equations being the simplest, and the NZS 4230:2004 [68] and TMS 402/602-16 [67] being the most complicated. TMS 402/602-16 also has added provisions to place limits on the calculated strength based on the $h_u/L_w$ ratio of a wall. Ranked in descending order, the shear-flexure capacity predictions were typically highest for NZS 4230:2004 [68], followed by Eurocode 6 [69], TMS 402/602-16 [67], and CSA S304-14 [17]. The median (MED) normalized capacity prediction for shear-flexure is 3.19, with a mean (MEAN) prediction of 3.21, a standard deviation (STD) of 0.63, and a coefficient of variation (COV) of 19.6%. These statistical values indicate that the code shear-flexure provisions have the best agreement among the capacity predictions. Based on the specimen behaviour, while diagonal cracking did occur in most of the specimens, none of the specimens exhibited a governing shear-flexure failure mode, which may be attributed to the large amount of horizontal reinforcement provided in the specimens.
5.3.2.3 Sliding

The sliding failure mode has shown somewhat higher normalized capacity predictions than the ductile flexural failure mode: the median (MED) normalized capacity prediction is 1.63, while the mean (MEAN) capacity is 1.59 with a standard deviation (STD) of 0.30 and a coefficient of variation (COV) of 18.6%. These statistical values would appear to indicate relatively good agreement among the codes, however high outlier capacity predictions by TMS 402/602-16 [67] for specimens W4 and W6 have been excluded (15.51 and 16.80, respectively). These notably high capacity predictions may be attributed to the TMS 402/602-16 [67] design provisions containing a prescribed allowable stress limit that acts over the area between the neutral axis and maximum compression fibre at the nominal moment capacity for walls with effective $h_d/L_w$ ratios greater...
than 1.0. These provisions, in conjunction with the tensile strain limits used to determine the size of the compression zone, result in potentially unconservative sliding capacity predictions in some cases. For walls with an effective $h_u/L_w$ ratio less than 1.0, the sliding capacity predictions in all codes are based on Coulomb-like friction theory, as discussed earlier in the chapter. The sliding capacity predictions are typically highest for TMS 402/602-16 [67], followed by NZS 4230:2004 [68], CSA S304-14 [17], and Eurocode 6 [69]. It should be noted that the highest code overpredictions were obtained for specimen W4. Despite initially experiencing ductile flexure behaviour during testing, the specimen eventually demonstrated a pronounced sliding failure that governed the response.

The sliding capacity predictions were on average 40 to 150% higher than the corresponding ductile flexural capacities; this is true for all codes except TMS 402/602-16 [67], which produced sliding capacity predictions that exceeded ductile flexural capacities by an even more significant margin. These results indicate that the sliding failure is less probable than a ductile flexural failure. Nonetheless, significant sliding displacements were recorded in six (out of eight) specimens during the testing, and in some cases contributed with as much as 35% of the total lateral displacement for high-ductility drift cycles (see Figure 72). Out of those six specimens, four (W3, W6, W7, and W8) initially experienced ductile flexural behaviour, which led to toe-crushing before the onset of significant sliding displacements. As the compression toes begin to experience toe-crushing and face shell spalling, some of the vertical reinforcing bars become exposed, which effectively eliminates their contribution to the shear-friction resistance mechanism. As a result, an increased propensity for sliding displacements is created. This agrees well with the values shown in Figure 82 and Figure 84. The capacity predictions indicated that a ductile flexural failure governs but sliding capacity predictions were not significantly higher for those specimens. Therefore, after toe-
crushing initiated, the outer vertical reinforcement was excluded from the sliding resistance and the sliding capacity was thus reduced by 20%, which agrees well with the experimental observations recorded for specimens W3, W6, W7, and W8.

Based on the test results and specimen behaviour, it appears that the sliding capacity provisions in the considered codes are generally adequate, provided that the specimens have only been subjected to a low number of lateral load cycles. However, after a larger number of cycles and a sustained flexural damage, sliding displacements may become larger than intended by the codes ([60]-[62]). Only TMS 402/602-16 commentary (Cl.9.3.6.5) addresses the effect of flexural damage on the sliding capacity by prescribing a slight reduction in the design coefficient of friction (from 0.7 to 0.65) [67].

![Figure 84. Code predictions for the sliding capacity normalized to the respective experimental results.](image-url)
5.3.3 Comparison of Explicit Lateral Displacement Code Provisions

Table 11 summarizes the key explicit code provisions for determining the lateral displacement capacities of DSRMWSs for each of the codes investigated in this chapter (TMS 402/602-16 [67], CSA S304-14 [17], NZS 4230:2004 [68], and EN 1996-1-1:2005/EN 1998-1:2004 [69], [70]). Since the elastic capacity of flexure-dominant elements such as DSRMSWs is accepted as corresponding to beam theory by all codes, Table 11 focuses on the inelastic capacity provisions for DSRMSWs, which largely equates to either the location of the plastic hinge and its permissible levels of inelastic rotation or the length of the compression zone. A comparison of the explicit displacement capacity provisions is then provided in a summary.
**Table 11. Lateral displacement capacity code provision summary.**

<table>
<thead>
<tr>
<th>Code</th>
<th>Requirement</th>
<th>Code Clause</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>TMS 402/602-16</td>
<td>$L_p = 0.5L_w$</td>
<td>Cl.C.3.1</td>
<td>$c =$ the distance from the maximum compressive fibre to the neutral axis based on a value of $P_u$ corresponding to Load Combination 5 of Section 2.3.2 of ASCE 7 [93], mm</td>
</tr>
<tr>
<td></td>
<td>$\theta_p = \frac{0.5L_w\varepsilon_{mu}}{c}$</td>
<td>Cl.C.3.1</td>
<td>$L_p =$ plastic hinge length/height, mm $L_w =$ wall length, mm $P_u =$ factored axial demand, N $\varepsilon_{mu} =$ maximum usable compressive strain of masonry, mm/mm $\theta_p =$ plastic hinge rotational deformation capacity, mm</td>
</tr>
<tr>
<td>CSA S304-14</td>
<td>$L_p = 0.5L_w + 0.1h_u$</td>
<td>Cl.16.9.4</td>
<td>$c =$ the distance from the maximum compressive fibre to the neutral axis, mm $h_u =$ unbraced height of the wall, mm $L_p =$ plastic hinge length/height, mm $L_w =$ wall length, mm $\varepsilon_{mu} =$ maximum usable compressive strain of masonry, mm/mm $\theta_p =$ plastic hinge rotational deformation capacity, mm</td>
</tr>
<tr>
<td></td>
<td>$\theta_p = \frac{\varepsilon_{mu}L_w}{2c} - 0.002$</td>
<td>Cl.16.8.8.3</td>
<td>$c =$ the distance from the fibre of maximum compressive strain to the neutral axis measured in a direction perpendicular to that axis, mm $L_p =$ plastic hinge length/height, mm $L_w =$ wall length, mm $\varepsilon_{mu} =$ maximum usable compressive strain of masonry, mm/mm $\theta_p =$ plastic hinge rotational deformation capacity, mm</td>
</tr>
<tr>
<td></td>
<td>$\theta_p \leq 0.025$</td>
<td>Cl.16.8.8.3</td>
<td>$c =$ the distance from the fibre of maximum compressive strain to the neutral axis measured in a direction perpendicular to that axis, mm $L_p =$ plastic hinge length/height, mm $L_w =$ wall length, mm $\varepsilon_{mu} =$ maximum usable compressive strain of masonry, mm/mm $\theta_p =$ plastic hinge rotational deformation capacity, mm</td>
</tr>
<tr>
<td>NZS 4230:2004</td>
<td>$L_p = minimum(L_w \frac{L_n}{6}, 600$ mm)</td>
<td>Cl.7.4.3</td>
<td>$c =$ the distance from the fibre of maximum compressive strain to the neutral axis measured in a direction perpendicular to that axis, mm $L_n =$ clear vertical distance between lines of effective horizontal support or clear horizontal distance between lines of effective vertical support, mm $L_p =$ plastic hinge length/height, mm $L_w =$ length of the wall in place of forces resisting the seismic loads, mm $T_1 =$ natural period of the first mode of vibration, s $\mu =$ structural ductility factor</td>
</tr>
<tr>
<td></td>
<td>$c \leq 0.4L_w \frac{L_n}{\mu}$</td>
<td>Cl.7.4.3</td>
<td>$L_p =$ plastic hinge length/height, mm $L_w =$ length of the wall in place of forces resisting the seismic loads, mm $\varepsilon_{mu} =$ maximum usable compressive strain of masonry, mm/mm $\theta_p =$ plastic hinge rotational deformation capacity, mm</td>
</tr>
<tr>
<td></td>
<td>$\mu = (4 &lt; 20(1 - T_1) &lt; 6)$</td>
<td>Cl.3.7.1.1</td>
<td>$L_p =$ plastic hinge length/height, mm $L_w =$ length of the wall in place of forces resisting the seismic loads, mm $\varepsilon_{mu} =$ maximum usable compressive strain of masonry, mm/mm $\theta_p =$ plastic hinge rotational deformation capacity, mm</td>
</tr>
<tr>
<td></td>
<td>$\theta_{u}\mu = \frac{1}{\gamma_{el}}(\theta_u + (\varphi_u - \varphi_i)L_{pl}(1 - \frac{0.5L_{pl}}{L_v}))$</td>
<td>Eurocode 8</td>
<td>$\theta_{u}\mu =$ plastic hinge length, mm $L_{pl} =$ plastic hinge length, mm $L_v =$ wall shear span, mm</td>
</tr>
<tr>
<td></td>
<td>$d_p = \frac{L_{pl}}{L_v}$</td>
<td>Eurocode 8</td>
<td>$d_p =$ longitudinal reinforcing bar diameter, mm $f'<em>m =$ specified masonry compressive strength, MPa $f_y =$ specified yield strength of steel reinforcement, MPa $h =$ member depth, mm $L</em>{pl} =$ plastic hinge length, mm $L_v =$ wall shear span, mm</td>
</tr>
</tbody>
</table>

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**Notes:**
- $L_p =$ plastic hinge length/height, mm
- $L_w =$ wall length, mm
- $P_u =$ factored axial demand, N
- $\varepsilon_{mu} =$ maximum usable compressive strain of masonry, mm/mm
- $\theta_p =$ plastic hinge rotational deformation capacity, mm
- $c =$ the distance from the maximum compressive fibre to the neutral axis
- $h =$ member depth, mm
- $L_{pl} =$ plastic hinge length, mm
- $L_v =$ wall shear span, mm
- $\gamma_{el} =$ elastic curvature factor
- $\varphi_u =$ ultimate curvature
- $\varphi_i =$ initial curvature
- $T_1 =$ natural period of the first mode of vibration
- $\mu =$ structural ductility factor
- $\mu = (4 < 20(1 - T_1) < 6)$
- $\theta_u =$ ultimate rotation capacity
- $\theta_p =$ plastic hinge rotational deformation capacity
- $d_p =$ longitudinal reinforcing bar diameter
- $f'_m =$ specified masonry compressive strength
- $f_y =$ specified yield strength of steel reinforcement
- $h =$ member depth
- $L_{pl} =$ plastic hinge length
- $L_v =$ wall shear span
- $\varepsilon_{mu} =$ maximum usable compressive strain
<table>
<thead>
<tr>
<th>Code</th>
<th>Requirement</th>
<th>Code Clause</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eurocodes 6 and 8 – EN 1996-1-1:2005/EN 1998-1:2004</td>
<td>For primary seismic elements: ( \gamma_{et} = 1.5 )</td>
<td></td>
<td>( L_{pl} ) = plastic hinge length, mm ( \gamma_{et} ) = seismic safety factor</td>
</tr>
<tr>
<td></td>
<td>For secondary seismic elements: ( \gamma_{et} = 1.0 )</td>
<td></td>
<td>( \phi_u ) = ultimate curvature at the end section, 1/mm ( \phi_y ) = yield curvature at the end section, 1/mm ( \theta_{um} ) = ultimate chord rotation capacity</td>
</tr>
</tbody>
</table>
As shown in Figure 85, the code provisions for predicting the lateral displacement capacity of DSRMSWs (defined here as the required lateral displacement capacity at the top of the wall needed to satisfy the corresponding “ductility check”-type provisions stipulated by each of the codes [36]) vary considerably between the different codes, primarily due to the critical flexural calculation differences summarized in Table 10. These prediction differences stand out when noting the high standard deviation (STD) of 1.26 and the extreme coefficient of variation (COV) of 97.0% for the examined codes. This is largely because the code displacement capacity predictions (often being linked to “ductility checks” [36]) are generally formulated with the assumption of flexure-dominant behaviour that leads into a toe-crushing failure mode, which as described earlier in the chapter, will often govern the ultimate lateral load capacity of flexure-dominant DSRMSWs. On an overall basis, however, the code displacement predictions did agree reasonably well with the experimental test displacements recorded at the end of the tests, as evidenced by the median (MED) normalized capacity prediction of 0.86 and the mean (MEAN) capacity of 1.16.

The code-provided equations for calculating the displacement capacities are similar for CSA S304-14 [17], TMS 402/602-16 [67], and Eurocode 6 [69] in that they require both a relatively small elastic component and a larger inelastic component to the total rotation and displacement. The NZS 4230:2004 [68] code provisions were based on limits on the compression zone length, which appeared to correspond very well with the experimental data, except for specimen W1, where the predicted capacity was significantly larger than predicted.

All of the codes provide equations for the plastic hinge length/height, which have a significant influence on the lateral displacement capacity. The plastic hinge length/heights, however, varied considerably among codes and in some cases by a factor of 3. The largest plastic
hinge lengths/heights are derived from the NZS 4230:2004 [68] provisions, the shortest are derived from the Eurocode 6 recommendations, and the CSA S304-14 [17] and TMS 402/602-16 [67] provisions recommend intermediate values. Limits on the masonry compression strain capacity also influenced the predicted displacement capacities; however the most significant contributors to the differences were the permissible tensile strain in the vertical reinforcement, which had a considerable effect on the compression zone length limits. This is the primary reason why there is a significant difference between the CSA S304-14 [17] and NZS 4230:2004 [68] displacement predictions when compared to the TMS 402/602-16 [67] and Eurocode 6 [69] predictions. These limits on the tensile strains in the vertical reinforcement and compression zone depth had a more pronounced effect on the code prediction differences for the T-shaped specimens (W6 and W7) and the axially-loaded specimen (W1). Since these two code-prescribed limits did not agree well with the test observations, the CSA S304-14 [17] and NZS 4230:2004 [68] displacement predictions were found to be generally closer to the experimental findings, whereas the TMS 402/602-16 [67] and Eurocode 6 [69] provide conservative lateral displacement capacity predictions.
5.3.4 **Comparison of Implicit Force-Based Code Provisions**

Table 12, Table 13, Table 14 summarize the key implicit force-based code provisions for DSRMSWs for bar buckling, rocking/bond slip, and lateral instability for each of the codes investigated in this chapter (TMS 402/602-16 [67], CSA S304-14 [17], NZS 4230:2004 [68], and EN 1996-1-1:2005/EN 1998-1:2004 [69], [70]). A comparison of the implicit force-based code provisions is then provided in summary.

**Figure 85.** Code predictions for the displacement capacity normalized to the respective experimental results.
Table 12. Bar-buckling code provision summary.

<table>
<thead>
<tr>
<th>Code</th>
<th>Requirement</th>
<th>Code Clause</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>TMS 402/602-16</td>
<td>Compression reinforcement may be included in the determination of flexural capacity if $s_t \leq \max {16d_l, 48d_t, \min(L_m, b_m, h_m)}$</td>
<td>Cl.9.3.2(e) and Cl.5.3.1.4</td>
<td>$b_m = \text{subject member width/thickness, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$d_l = \text{diameter of longitudinal reinforcing bars, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$d_t = \text{diameter of reinforcing tie bars, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$h_m = \text{subject member height, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$L_m = \text{subject member length, mm}$</td>
</tr>
<tr>
<td>CSA S304-14</td>
<td>Compression reinforcement may be included in the determination of flexural capacity if $s_t \leq \max {16d_l, 48d_t, \min(L_m, b_m, h_m)}$ and $d_t \geq 3.65 \text{ mm}$</td>
<td>Cl.10.2.7 and Cl.12.2.1</td>
<td>$b_m = \text{subject member width/thickness, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$d_l = \text{diameter of longitudinal reinforcing bars, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$d_t = \text{diameter of reinforcing tie bars, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$h_m = \text{subject member height, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$L_m = \text{subject member length, mm}$</td>
</tr>
<tr>
<td>NZS 4230:2004</td>
<td>No explicit reference to compression steel or bar-buckling</td>
<td>Not applicable</td>
<td>Not applicable</td>
</tr>
<tr>
<td>Eurocodes 6 and 8 – EN 1996-1-1:2005/EN 1998-1:2004</td>
<td>No explicit reference to compression steel or bar-buckling</td>
<td>Not applicable</td>
<td>Not applicable</td>
</tr>
</tbody>
</table>
Table 13. Rocking/bond-slip code provision summary.

<table>
<thead>
<tr>
<th>Code</th>
<th>Requirement</th>
<th>Code Clause</th>
<th>Remarks</th>
</tr>
</thead>
</table>
| TMS 402/602-16          | $l_d \geq \frac{0.13d_b^2f_y}{K\sqrt{f'_m}}$                               | Cl.6.1.5.1.1| $d_b$ = reinforcing bar diameter, mm
|                         |                                                                               | Eq. 6-1     | $f'_m$ = specified masonry compressive strength, MPa
|                         |                                                                               |             | $f_y$ = specified yield strength of steel reinforcement, MPa
|                         |                                                                               |             | $K \leq$ (minimum masonry cover; clear spacing between adjacent reinforcement splices; 9$d_b$)
|                         |                                                                               |             | $l_d$ = development length of reinforcing bar in anchorage
|                         |                                                                               |             | $\gamma = 1.0$ for M10-M16 bars; 1.3 for M19-M22 bars; and 1.5 for M25 and larger bars |
| CSA S304-14             | $l_d = 1.15 \frac{k_1k_2k_3 f_y}{(d_{cs} + K_{tr}) \sqrt{f'_gr}} A_b$        | Cl.12.4.2.3 | $A_b$ = area of reinforcing bar, mm$^2$
|                         | $K_{tr} = \frac{A_{tr} f_{yt}}{10.5 \pi n}$                                |             | $A_{tr} =$ area of transverse reinforcing bars, mm$^2$
|                         | $(d_{cs} + K_{tr}) \leq 2.5d_b$                                             |             | $d_b$ = nominal diameter of reinforcing bar, mm
|                         |                                                                               |             | $f'_gr$ = specified grout compressive strength, MPa
|                         |                                                                               |             | $f_y$ = yield strength of reinforcement, MPa
|                         |                                                                               |             | $f_{yt}$ = yield strength of transverse reinforcement, MPa
|                         |                                                                               |             | $k_1 =$ (i) 1.3 for horizontal reinforcement so placed that more than 300 mm of fresh grout is cast in the member below the development length or splice; (ii) 1.0 for other cases (Cl.12.4.2.5a)
|                         |                                                                               |             | $k_2 =$ (i) 1.5 for epoxy-coated reinforcement with clear cover less than 3$d_b$, or with clear spacing between bars being developed less than 6$d_b$; (ii) 1.0 for all other epoxy-coated reinforcement; and (iii) 1.0 for uncoated reinforcement (Cl.12.4.2.5b)
|                         |                                                                               |             | $k_3 =$ (i) 0.8 for $d_b \leq 20$ mm; (ii) 1.0 for $d_b > 20$ mm (Cl.12.4.2.5c)
|                         |                                                                               |             | $l_d$ = development length of deformed reinforcing bars, mm
|                         |                                                                               |             | $n =$ numbers of bars being spliced or developed along the potential plane of bond splitting
|                         |                                                                               |             | $s =$ maximum spacing of transverse reinforcing within $l_d$, mm
<p>|</p>
<table>
<thead>
<tr>
<th>Code</th>
<th>Requirement</th>
<th>Code Clause</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>NZS 4230:2004</td>
<td>( L_{db} = 40d_b ) for ( f_y = 300 \text{ MPa} )</td>
<td>Cl.6.3.7.3</td>
<td>( A_{sr} = \text{area of flexural reinforcement required, mm}^2 ) ( A_{sp} = \text{area of flexural reinforcement provided, mm}^2 ) ( d_b = \text{reinforcing bar diameter, mm} ) ( f_y = \text{specified yield strength of steel for reinforcement, MPa} ) ( MF = \text{product of applicable modification factors: (a) 1.3 for top horizontal reinforcing bars where more than 300 mm of fresh grout is cast in the component below the bar; (b) 1.0 for reinforcing bars that do not contain an expansive admixture as defined in NZS 4210; (c) } A_{sl}/A_{sp} \text{ for reinforcing bars outside of critical sections of components subjected to earthquake loads} )</td>
</tr>
<tr>
<td></td>
<td>( L_{db} = 70d_b ) for ( f_y = 500 \text{ MPa} )</td>
<td>Cl.6.3.7.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( L_d = L_{db}MF )</td>
<td>Cl.6.3.7.5</td>
<td></td>
</tr>
<tr>
<td>Eurocodes 6 and 8</td>
<td>( l_b = \frac{\varphi}{\gamma_s} f_{ym} \frac{1}{f_{bom}} )</td>
<td>Cl.8.2.5.1(6)(i)</td>
<td>( f_{bom} = \text{design anchorage bond strength of reinforcing steel, MPa} ) ( f_{bok} = \text{characteristic anchorage bond strength of reinforcing steel, MPa} ) Table 3.5 ( f_{ym} = \text{design strength of reinforcing steel, MPa} ) ( l_b = \text{straight anchorage length for reinforcing bars with constant bond stress, mm} ) Where a greater area of reinforcing steel is provided than is required by design, the anchorage length may be reduced proportionally provided that reinforcing steel in tension is not less than the greater of ( 0.3l_b, 10\varphi, ) or ( 100 \text{ mm} ) (Cl.8.2.5.1(6)(i)) ( \varphi = \text{effective diameter of the reinforcing steel, mm} ) ( \gamma_m = \text{partial safety factor for masonry properties} ) ( \gamma_s = \text{partial safety factor for steel} )</td>
</tr>
</tbody>
</table>
Table 14. Lateral instability code provision summary.

<table>
<thead>
<tr>
<th>Code</th>
<th>Requirement</th>
<th>Code Clause</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>TMS 402/602-16</td>
<td>No specific limits; however, limits exist on compressive stress exist</td>
<td>Cl.9.2.4.2</td>
<td>$A_n = \text{net cross-sectional area of the member, mm}^2$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Eq. 9-11</td>
<td>$f'_m = \text{specified masonry compressive strength, MPa}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cl.9.3.5.4.2</td>
<td>$P_u = \text{factored axial demand, N}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$h = \text{unsupported height of a wall, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$t = \text{thickness of a wall or column, taking into account any reduction in thickness, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\frac{h}{t} &gt; 30$, $\frac{P_u}{A_n} \leq 0.05f'_m$</td>
</tr>
<tr>
<td>CSA S304-14</td>
<td>$h + 10 \leq 16$</td>
<td>Cl.16.9.3.3</td>
<td>$h = \text{unsupported height of a wall, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$t = \text{thickness of a wall or column, taking into account any reduction in thickness, mm}$</td>
</tr>
<tr>
<td>NZS 4230:2004</td>
<td>General minimum wall thickness in plastic hinge regions is</td>
<td>Cl.7.4.4.1</td>
<td>$A_{sr} = \text{area of flexural reinforcement required, mm}^2$</td>
</tr>
<tr>
<td></td>
<td>$b \geq 0.075L_n$</td>
<td></td>
<td>$A_{sp} = \text{area of flexural reinforcement provided, mm}^2$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$b = \text{thickness of wall section, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$c = \text{the distance from the fibre of maximum compressive strain to the neutral axis}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>measured in a direction perpendicular to that axis, mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$L_f = \text{length of wall flange/return, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$L_n = \text{clear vertical distance between lines of effective horizontal support or clear}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\text{horizontal distance between lines of effective vertical support, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$\frac{h}{t} \leq 27$</td>
</tr>
<tr>
<td>Eurocodes 6 and 8 – EN 1996-1-1:2005/EN 1998-1:2004</td>
<td>$h_{ef} / t_{ef} \leq 27$</td>
<td>Cl.6.1.6</td>
<td>$h_{ef} = \text{effective height of wall flange, mm}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$t_{ef} = \text{effective thickness of wall flange, mm}$</td>
</tr>
</tbody>
</table>
Following the explicit code provision comparisons discussed in the previous sections, the implicit code provisions are discussed in this section for the bar-buckling, rocking/bond slip, and lateral instability failure modes. Implicit provisions related to each of these failure modes are comparatively short and simple.

Only TMS 402/602-16 [67] and CSA S304-14 [17] contain provisions for preventing bar buckling, which are in the form of maximum tie spacing around the vertical reinforcement at the wall end zones. The maximum tie spacing is calculated in almost an identical manner according to these two codes. If these provisions are satisfied, the designer realizes the benefit of accounting for the compression reinforcement in the determination of flexural capacity. Neither NZS 4230:2004 [68] nor Eurocode 6 [69] contain provisions regarding the bar buckling or tied compression reinforcement.

Rocking/bond-slip provisions are provided in all codes in the form of minimum development/splice lengths. The complexity of the development/splice length calculations varies significantly between the codes. Both CSA S304-14 [17] and TMS 402/602-16 [67] calculate development/splice length based on several factors, but the CSA S304-14 [17] calculations are more comprehensive. The Eurocode 6 [69] development/splice length calculations are solely based on bar diameter, whereas the NZS 4230:2004 [68] prescribes splice length calculations based on both bar diameter and a product of modification factors. Due to the relative simplicity of the Eurocode 6 [69] and NZS 4230:2004 [68] provisions, these estimates are expected to produce more conservative results than CSA S304-14 [17] and TMS 402/602-16 [67]. Moreover, in all of the codes except Eurocode 6 [69], there are also limits on the minimum vertical reinforcement ratio or bar size, which can implicitly help designers prevent rocking due to a potential lack of flexural crack redistribution.
Lateral instability is addressed by all codes in an implicit form (through \( \frac{h_u}{t_w} \) restrictions). However, TMS 402/602-16 [67] addresses lateral instability by setting the compression stress limits for walls with high \( \frac{h_u}{t_w} \) ratios, indicating that these limits are more related to the standard (Euler) buckling caused by uniaxial compression – as opposed to the lateral instability failure mode described earlier in this chapter. It is also unclear whether the \( \frac{h_u}{t_w} \) limits prescribed by Eurocode 6 are related to Euler buckling or lateral instability since lateral instability is associated with seismic loading and Eurocode 6 [69] is related to the design of non-seismic loading actions. Both CSA S304-14 [17] and NZS 4230:2004 [68] prescribe \( \frac{h_u}{t_w} \) limits that are dependent on the expected ductility demand and are likely to be associated with preventing lateral instability in ductile reinforced masonry shear walls.

5.4 Summary and Conclusions

This chapter provided an overview of the most prominent in-plane flexure-related failure modes affecting DSRMSWs, including ductile flexural, shear-flexure, sliding, toe-crushing, bar-buckling, rocking/bond-slip, and lateral instability. The seismic code provisions for DSRMSWs from several international masonry design codes were used for a comparison of the nominal code-predicted ULS lateral load capacities and the experimental values obtained from the experimental phase of this study. The following conclusions can be derived from this investigation:

All of the DSRMSW specimens exhibited ductile flexural failure mode in the early stages of the tests; however, it was found that it is common for this failure mode to eventually transition to toe-crushing, which governed the lateral load capacity of most of the specimens. Ductile flexural behaviour initiated with flexural yielding of the vertical reinforcement, which typically occurred
at a lateral drift close to 0.20%, whereas toe-crushing usually didn’t occur until 0.50% to 1.20% drift.

The remaining flexure-dominant failure modes generally occurred only after the ductile-flexural and toe-crushing failure modes initiated. Aside from specimen W4, sliding occurred at drifts ranging from 0.15% to 2.15% drift, and rocking occurred at drifts ranging from 0.19% to 3.24%. Shear-flexural behaviour only occurred in specimen W1 at a drift of 0.29%, whereas lateral instability in all affected specimens initiated at drifts around 1.6%. Bar fracture was found to occur at the highest drift demands, which ranged from 1.27% to 3.33%.

By comparing Figure 82, Figure 83, and Figure 84, it can be seen that CSA S304-14 [17], NZS 4230:2004 [68], and Eurocode 6 [69] generally predict the highest normalized force capacities for shear-flexure; whereas TMS 402/602-16 [67], the sliding capacity predictions are the highest. Ductile flexural capacity predictions were generally the lowest, which is expected considering that this was effectively the intent of the specimen design.

Ductile flexural capacity predictions were found to follow basic beam theory for all codes; however, the predicted capacities did fit closer with the test data when tensile strain restrictions on the reinforcement were disregarded. Note that CSA S304-14 [17] and NZS 4230:2004 [68] do not include these strain limits, whereas TMS 402/602-16 [67] and Eurocode 6 [69] do. Tensile strain limits have a beneficial effect of decreasing the chances of brittle secondary flexure-related failure modes, such as bar-buckling and lateral instability, which primarily occur at higher ductility demands.

Shear-flexural capacity predictions from all of the studied codes had a similar methodology of combining the contributions of masonry, horizontal reinforcement, and axial compression to the total shear-flexure resistance mechanism. Although the horizontal reinforcement provided in the
test specimens was higher than for a DSRMSW in typical practical applications, the code provisions appeared to have been adequate in preventing a shear-flexural failure mode from occurring in any of the specimens. Overall, shear-flexure capacity predictions also had the best agreement among the codes.

Sliding capacity predictions for the codes that use Coulomb-like friction as their basis, for all types of DSRMSWs, were found to fit better with the test data than codes that incorporate prescribed allowable stress that acts over the compression zone area of the wall (TMS 402/602-16 [67]). In all cases, the code provisions appeared to provide the most accurate predictions when the number of lateral load cycles was low (less than 5). Sliding displacements were found to significantly when the number of lateral load cycles became high (more than 8). This may be attributed to a combination of factors, including loss of surface roughness along with the sliding interface; increased inelastic tensile strains in the vertical reinforcement, and the effect of the prior occurrence of toe-crushing and flexural damage on reducing the sliding capacity of the wall. Thus, there may be a need to revise code provisions for walls expected to experience high-ductility cyclic loading demands. Some potential simple adjustments could involve a decrease in the coefficient of friction for the Coulomb-like friction mechanism (in part already covered by TMS 402/602-16 [67]), as well as a reduction of the clamping force provided by the vertical reinforcement.

Displacement predictions were found to be sensitive to the code prescriptions for plastic hinge length/height and limits to the masonry compression strain, tensile strains in the vertical reinforcement, and the length of the compression zone. Considering the significant differences between the codes for these parameters, it is understandable that the code displacement predictions varied considerably. Based on the test specimens examined in the experimental study, the codes
that did not include limits on the vertical reinforcement strains were found to produce displacement predictions that agreed better with the experimental findings.

Lateral instability is intrinsically related to displacement ductility demands, but it is implicitly covered only by NZS 4230:2004 [68] and CSA S304-14 [17] through $h_u/t_w$ limits. The $h_u/t_w$ restrictions contained in the TMS 402/602-16 [67] and Eurocode 6 [69] $h_u/t_w$ appear to be related to Euler buckling (due to the effect of increasing axial loading), as opposed to lateral instability due to in-plane lateral loading. Since Class C lateral instability was not observed in any of the tests, it is difficult to determine the effectiveness of $h_u/t_w$ limits or other restrictions based on the lateral instability failure mode.

Overall, this chapter has shown that the explicit provisions on the lateral load capacity of DSRMSWs can vary significantly between the leading masonry design codes investigated in this chapter, with CSA S304-14 [17] providing good lateral load capacity predictions compared to the other investigated codes. Significant differences in the various methodologies among the codes were found to lead to significant discrepancies in the predicted lateral load capacities. These differences would seem to help encourage the trend toward PBD provisions in new editions of masonry design codes and the increased use of accurate nonlinear models in standard design practices. The next chapter describes one type of model that has not previously been implemented for DSRMSWs but has considerable advantages over other similar modelling approaches.
Chapter 6: Analytical Model

6.1 Introduction
This chapter begins with a description of the MVLEM mechanics, which is followed by parametric analyses to calibrate the critical hysteretic parameters that influence the behaviour of MVLEs at the microscopic level. The parametric analyses are then used to determine recommended values for the MVLE hysteretic parameters by verification of the results using the results of the eight full-scale DSRMSW specimens tested in the experimental phase of this study. After determining the recommended hysteretic parameters from the parametric analyses at the microscopic level, a sensitivity study is conducted to determine a recommended number and distribution of stacked MVLEs to represent a physical wall at the macroscopic level, as well as a recommended level of spring discretization for each MVLE at the macroscopic level. The same eight DSRMSW test specimens from the experimental phase of this study are used in the verification of the sensitivity study. The results of the parametric and sensitivity studies help provide a set of recommended modelling approaches when using MVLEs to determine the displacement-based performance of DSRMSWs. These recommended values are then used in a model validation to numerically predict the lateral load-displacement results of three experimentally-tested DSRMSW specimens, conducted by Shhid [94], in order to verify the modelling approach.

6.2 Model Composition
The original MVLEM by Kabeyasawa et al. in 1983 [48] consisted of a rotational and horizontal spring at the centre of the modelled wall, as well as three vertical springs – one at each of the wall end zones and one at the centre of the wall – which spanned between two rigid horizontal beams
at the top and bottom of the element. With the subsequent development of the model, the horizontal spring remained, but the central rotational spring was replaced by additional central vertical springs over the length of the modelled physical wall [49]-[56]. This is shown diagrammatically in Figure 86, where \( L \) is the element length/height, \( L \cdot \kappa \) is the relative height of the horizontal spring, and \( \Phi_{\text{top}}, \Phi_{\text{bot}}, \) and \( \Delta_{\text{axial}} \) represent the respective degrees of freedom of the element. Each of the springs represents a discretized cross-sectional segment of the modelled physical wall’s total cross-section area, as illustrated in Figure 87. The characteristics of these springs are dependent on the strength and area of the base material (e.g. concrete or masonry), and reinforcement (e.g. steel) in each cross-sectional wall segment and each wall segment must be capable of resisting some degree of both compression and tension to function correctly. In this regard, the MVLEM is similar to a fibre model, with the main difference between the two model types being that fibre models deal with stresses and strains of the base material and reinforcement fibres, whereas the MVLEM deals with the forces and elongations of the springs. The vertical springs are effectively lumped at the centroid of each wall segment and are used to capture the axial and flexural behaviour of the modelled wall by using simple hysteretic rules based on a set of experimentally-calibrated hysteretic parameters that are described in Figure 88: \( \alpha, \beta, \delta, \) and \( \gamma \). Table 15 describes these hysteretic parameters as well as the other defining parameters that are used to describe the vertical spring characteristics shown in Figure 88 and the horizontal spring characteristics shown in Figure 89. The horizontal spring is primarily defined by experimentally-derived shear-slip displacements and forces, which when experimental data is not available, were found to be reasonably approximated by assuming an initial stiffness, \( k_{\text{horiz}} \), as shown in Table 15 and a shear-slip yield force, \( Q_y \), equal to 90% of the unfactored Columb-like friction equations typically prescribed in
masonry design code provisions (e.g. [17]). Additionally, horizontal spring characteristics are also reliant on the experimentally calibrated hysteretic parameters of $\kappa$ and $\eta$.

While the MVLEM currently has the capacity to model the ductile-flexure, shear-flexure, sliding, and toe-crushing failure modes, it is currently not well set up to capture the rocking or lateral instability failure modes. To capture rocking, the hysteretic force-displacement relation for the vertical springs would need to be modified to provide a limit on the maximum tensile force prior to bond-slip and bar-fracture. This is a difficult task for bar-fracture in particular since this mechanism is also dependent on the sliding displacement history during the test, as discussed in the previous chapter. Since this would require a great deal of supporting experimental evidence, of which is currently not present, the effect of rocking on the lateral load-displacement response is ignored – potentially leading to some discrepancy in the modelled responses of test specimens that were observed to exhibit this behaviour. Similarly, the perceived effect of out-of-plane displacements reducing the lateral stiffness of the wall is ignored due to an absence of supporting experimental evidence. Moreover, significant modification of the hysteretic force-displacement relation for the vertical springs would need to be carried out to capture the effect of the lateral instability failure mode occurring following a tensile strain excursion that exceeded a precalculated limit, but only if other failure influencing modes (e.g. sliding or toe-crushing) did not initiate to an extent that would preclude lateral instability from occurring, as discussed in the previous two chapters. Since the effect of out-of-plane displacements and the lateral instability failure mode are also ignored, test specimen responses exhibiting these behaviour types will also likely show variance from the modelled response.
Figure 86. MVLE component layout.

Figure 87. Physical wall cross-section discretized into a series of vertical springs in the numerical model.
Figure 88. MVLE hysteretic force-displacement relation for typical vertical springs (adapted from Fischinger et al. [54]).

Figure 89. MVLE hysteretic force-displacement relation for typical horizontal springs.
Table 15. Determination of MVLE spring parameters.

<table>
<thead>
<tr>
<th>Spring Type</th>
<th>Defining Parameters</th>
<th>Parameter Description</th>
<th>Derivation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical</td>
<td>$k_1$</td>
<td>Elastic Vertical Spring Stiffness</td>
<td>$A_bE_b/L$</td>
</tr>
<tr>
<td></td>
<td>$F_{cr}$</td>
<td>Cracking Force of Base Material</td>
<td>$F_{cr}/f_bA_b$</td>
</tr>
<tr>
<td></td>
<td>$F_y$</td>
<td>Yield Force of Vertical Reinforcement</td>
<td>$F_y/f_rA_s$</td>
</tr>
<tr>
<td></td>
<td>$\Delta_y$</td>
<td>Vertical Spring Elongation at Yield Force</td>
<td>$F_yL/E_sA_s$</td>
</tr>
<tr>
<td></td>
<td>$k_2$</td>
<td>Slope between Initiation of Reloading Curve and Yield of Vertical Reinforcement</td>
<td>$F_yk_1(\beta+1)/(8F_y+k1\Delta_y)$</td>
</tr>
<tr>
<td></td>
<td>$k_3$</td>
<td>Hardening Stiffness of Vertical Reinforcement</td>
<td>$f_{hard}k_s$</td>
</tr>
<tr>
<td></td>
<td>$k_4$</td>
<td>Initial Slope of Unloading Curve</td>
<td>$k_2(\Delta_y/\Delta_{max})^\delta$</td>
</tr>
<tr>
<td></td>
<td>$F_b$</td>
<td>Maximum Compression Force of Base Material</td>
<td>$f_bA_b$</td>
</tr>
<tr>
<td></td>
<td>$\Delta_b$</td>
<td>Vertical Spring Contraction at Maximum Compression Force</td>
<td>$\varepsilon_bL$</td>
</tr>
<tr>
<td></td>
<td>$F_{bu}$</td>
<td>Residual Compression Force (Following Base Material Crushing)</td>
<td>$f_{bu}A_b$</td>
</tr>
<tr>
<td></td>
<td>$\Delta_{bu}$</td>
<td>Vertical Spring Contraction at Residual Compression Force</td>
<td>$\varepsilon_{bu}L$</td>
</tr>
<tr>
<td></td>
<td>$\Delta_{max}$</td>
<td>Maximum Displacement of Loading Previous Cycle</td>
<td>Model Dependent</td>
</tr>
<tr>
<td></td>
<td>$\alpha$</td>
<td>Related to Length of the Tension Unloading Curve in Terms of Force</td>
<td>Experimental Calibration</td>
</tr>
<tr>
<td></td>
<td>$\beta$</td>
<td>Initiation of the Compression Loading Curve in Terms of Force</td>
<td>Experimental Calibration</td>
</tr>
<tr>
<td></td>
<td>$\gamma$</td>
<td>Related to the Post-Yield Stiffness of the Tension Loading Curve</td>
<td>Experimental Calibration</td>
</tr>
<tr>
<td></td>
<td>$\eta$</td>
<td>Related to Slope of the Tension Unloading Curve</td>
<td>Experimental Calibration</td>
</tr>
<tr>
<td></td>
<td>$Q_y$</td>
<td>Horizontal Spring Force at Shear-Slip Yielding</td>
<td>Experimental/Code Values</td>
</tr>
<tr>
<td>Horizontal</td>
<td>$k_{horiz}$</td>
<td>Elastic Horizontal Spring Stiffness</td>
<td>$A_{br}G_b/L$</td>
</tr>
<tr>
<td></td>
<td>$d_y$</td>
<td>Horizontal Spring Elongation at Shear-Slip Yielding</td>
<td>$Q_y/k_{horiz}$</td>
</tr>
<tr>
<td></td>
<td>$\eta$</td>
<td>Related to Length of the Shear-Slip Unloading Curve in Terms of Force</td>
<td>Experimental Calibration</td>
</tr>
<tr>
<td></td>
<td>$\kappa$</td>
<td>Normalized Length/Height of Horizontal Spring</td>
<td>Experimental Calibration</td>
</tr>
</tbody>
</table>

Notes:
- $A_b$: Base Material Cross-Sectional Area in Segment
- $A_{br}$: Base Material Cross-Sectional Area of Segment in Shear
- $A_r$: Reinforcement Cross-Sectional Area in Segment
- $E_b$: Elastic Modulus of Base Material
- $E_r$: Elastic Modulus of Reinforcement
- $f_b$: Compressive Strength of Base Material
- $f_{br}$: Residual Strength of Base Material
- $f_{cr}$: Tensile Strength of Base Material
- $f_y$: Yield Strength of Base Material
- $G_b$: Elastic Shear Modulus of Base Material
- $L$: MVLE Length/Height
- $\varepsilon_b$: Compressive Strain of Base Material
- $\varepsilon_{bu}$: Ultimate Compressive Strain of Base Material
6.3 Model Calibration and Sensitivity Analyses

As shown in Table 15, many of the defining variables for the vertical and horizontal springs of the MVLEM are derived from equations based on well-known properties of the constituent materials; however the hysteretic parameters of the vertical and horizontal springs must be calibrated from experimental tests. Although these hysteretic parameters have been well-calibrated for RCSWs (e.g. $\alpha \approx 1.0$, $\beta \approx 1.5$, $\delta \approx 0.5$, $\gamma \approx 1.05$, $\kappa \approx 0.3$, and $\eta \approx 0.2$ [53]-[56]), they have not been calibrated for RMSWs or DSRMSWs since the MVLEM approach has not yet been used to model these types of walls. These parameters are fundamental to the behaviour of MVLEMs and thus need to be calibrated on a number of experimental tests, as is carried out in Section 6.3.1.

Moreover, similar to a displacement-based fibre element, the MVLEM is also dependent on the discretization of the springs within the physical cross-section being modelled, as well as the number of elements over the height of the modelled physical wall. Generally, a higher degree of spring discretization will yield more accurate results and increase the likelihood of convergence for modelled walls that have displacement demands that exceed the ultimate capacity. The drawback to a higher degree of spring discretization is that modelling effort and computational time also increase with the increasing number of springs. Similarly, since MVLEs are displacement-based elements, a higher degree of element discretization, i.e. an increasing number of elements over the height of a modelled wall, will tend to improve the model accuracy, however also at a cost in terms of modelling effort and computational time. It is therefore important to determine a suitable compromise between the level of element and spring discretization and the modelling effort and computational time – Section 6.3.2 helps to determine this compromise for DSRMSW MVLEMs.
6.3.1 Model Calibration

This section of the chapter describes the parametric analyses carried out to calibrate the key hysteretic parameters that influence the behaviour of MVLEs at the microscopic level. The first set of parametric analyses investigate the four key hysteretic parameters of the MVLE vertical springs, i.e. $\alpha$, $\beta$, $\delta$, and $\gamma$, as described in the previous section of the chapter. The second set of parametric analyses investigate two key hysteretic parameters of the MVLE horizontal springs, i.e. $\kappa$ and $\eta$. All of the parametric analyses used MVLEMs simulating the experimental testing of the eight full-scale DSRMSW test specimens described in the experimental phase of this study for verification. The methodology of the parametric analyses involved first determining a typical range of hysteretic parameter values that produced MVLEM responses that demonstrated good overall agreement with the experimental behaviour, with specific focus on the backbone envelope points of the experimental and numerical hystereses, as described in Section 4.3.1.1, since these values are most important in the currently-prevalent force-based design methodologies in place in CSA S304-14. This was conducted by selecting parametric and sensitivity values that produced the lowest average discrepancy between the experimental and numerical hystereses for all of the backbone envelope points. This exercise was carried out for each of the test specimens and involved numerous iterations to refine the hysteretic parameters as much as possible. The modelling was carried out in the OpenSees software framework [95] using a base model initially developed by Fischinger et al. [54]. The modelled results of MVLEMs using the low end, middle, and high end values of each hysteretic parameter range are plotted with the experimental results from the test data from specimen W2, which was generally representative of the other specimens from the experimental phase of this study in terms of the effects of variability in the hysteretic parameters. The models used a “medium” level of element and spring discretization as described
in the next section of the chapter, which is focused on the macroscopic-level parametric analysis. This chapter of the dissertation concludes with a summary of hysteretic parameters found to produce the best agreement between the model and experimental results for each test specimen, as well as a set of recommended values for each of these parameters.

6.3.1.1 Vertical Spring Parametric Analyses

6.3.1.1.1 Hysteretic Parameter $\alpha$

The hysteretic parameter $\alpha$ affects the length of the unloading curve in terms of force for the vertical spring hysteretic force-displacement relation, as shown in Figure 88. In the global model, changes in $\alpha$ result in differences in the unloading curves, as shown in Figure 90. Lower values of $\alpha$ tend to shorten the length of the initial unloading slope, whereas higher significant tend to extend the length. These effects are found to have more influence at higher drift levels (exceeding 1%) since the unloading curves are longer. At low to medium drift levels up to 1% drift, the effects of $\alpha$ are relatively insignificant. The MVLEMs were generally found to be capable of accommodating only low variances in $\alpha$ without encountering convergence issues.
6.3.1.1.2 Hysteretic Parameter $\beta$

The hysteretic parameter $\beta$ affects the initiation of the compression loading curve in terms of force for the vertical spring hysteretic force-displacement relation, as shown in Figure 88. In the global model, changes in $\beta$ primarily result in differences at the transition between the unloading and reloading curves, as shown in Figure 91. The $\beta$ parameter, however, also has a small effect on the initial slope of the unloading curve. Increasing values of $\beta$ tend to initiate the reloading curve at rising levels of drift, thus shortening the unloading curves overall. Increasing $\beta$ values also tend to slightly reduce the initial slope of the unloading curve, as well as precipitating toe-crushing at slightly lower drift levels. The MVLEM was found to accommodate comparatively significant variations in $\beta$ without encountering convergence issues.
6.3.1.1.3 Hysteretic Parameter $\delta$

The hysteretic parameter $\delta$ affects the slope of the tension unloading curve for the hysteretic force-displacement relation for vertical springs, as shown in Figure 88. In the global model, changes in $\delta$ result in differences in the unloading curves, as shown in Figure 92. Lower values of $\delta$ tend to increase the gradient of the initial unloading slope, whereas high values tend to decrease its gradient. These effects are found to have more influence at moderate to moderate- to high-drift levels over 0.2% drift, whereas $\delta$ has little impact on the model for drift levels below 0.2%. The MVLEM was found to accommodate comparatively significant variations in $\delta$ without encountering convergence issues.
Figure 92. Parametric study summary for hysteretic parameter $\delta$ (only specimen W2 summary shown).

6.3.1.1.4 **Hysteretic Parameter $\gamma$**

The hysteretic parameter $\gamma$ affects the post-yield stiffness of the tension reloading curve for the vertical spring hysteretic force-displacement relation, as shown in Figure 88. In the global model, changes in $\gamma$ result in small only differences in the reloading curves, as shown in Figure 93. Lower values of $\gamma$ tend to slightly increase the gradient of the initial reloading slope as well as the length of the second reloading slope, whereas high values of $\gamma$ have the opposite effect. Changes in $\gamma$ were found to have the most significant impact on the wall response for drifts exceeding 0.5%. The MVLEM was found to be highly sensitive to variations in $\gamma$, with values outside the range examined in Figure 93 typically causing convergence issues.
6.3.1.2 Horizontal Spring Parametric Analyses

6.3.1.2.1 Hysteretic Parameter $\kappa$

The hysteretic parameter $\kappa$ affects the normalized length/height of the horizontal springs, as shown in Figure 86. In the global model, changes in $\kappa$ result in differences in the unloading curves, but generally only after toe-crushing has initiated, as shown in Figure 94. Lower values of $\kappa$ tend to precipitate toe-crushing at mildly lower drift levels and affect the subsequent unloading curves. These effects are found to have more influence at higher drift demands since the unloading curves are longer. The MVLEM was found to accommodate only modest variations in $\kappa$ without running into convergence issues.
6.3.1.2.2 Hysteretic Parameter $\eta$

The hysteretic parameter $\eta$ affects the length of the shear-slip unloading curves for the hysteretic force-displacement relation for horizontal springs shown in Figure 89. In the global model, changes in $\eta$ result in moderate differences in the unloading curves as well as small differences in the reloading curves of the hysteretic response, as shown in Figure 95. Lower values of $\eta$ tend to shorten the length of the initial unloading slope, whereas higher values tend to extend the length. Lower values of the parameter $\eta$ also mildly delay the onset of the reloading curve. Variances in the parameter $\eta$ affect global response nearly equally at all drift levels. The MVLEM was found to accommodate only mild variations in $\eta$ without running into convergence issues.
Figure 95. Parametric study summary for hysteretic parameter $\eta$ (only specimen W2 summary shown).

### 6.3.1.3 Recommended Hysteretic Parameter Values

Table 16 provides a summary of recommended values for the hysteretic parameters investigated in this section of the chapter for MVLE vertical and horizontal springs. It should be noted that these recommended values are based only on the experimental tests on the specimens from the experimental phase of this study and that further investigation is required to confirm these recommendations. The recommended values were determined through identification of the best overall agreement with the experimental results following numerous iterations. The mean values were found to vary from the typical values used for RCSWs (e.g. $\alpha \approx 1.0$, $\beta \approx 1.5$, $\delta \approx 0.5$, $\gamma \approx 1.05$, $\kappa \approx 0.3$, and $\eta \approx 0.2$) and are later employed for the validation of the model using the experimental test results recorded by Shedid [94]. As shown in Table 16, while there found to be low coefficients of variation (COV) for the $\alpha$, $\gamma$, $\kappa$, and $\eta$ hysteretic parameter values, the COVs for $\beta$ and $\delta$ were found to be much higher. This leads to the conclusion that while most of the hysteretic parameters
are not strongly affected by the different DSRMSW design parameters provide in Table 1, the $\beta$ and $\delta$ hysteretic parameters are influenced by varying DSRMSW design parameters. As a result, separate analysis of the $\beta$ and $\delta$ hysteretic parameters was carried out to develop expressions for the values of these hysteretic parameters based on changes of each of the DSRMSW design parameters. The findings from this analysis showed that the $\beta$ hysteretic parameter was primarily only affected by changes in the $h_w/t_w$ ratio and the vertical reinforcement ratio, $\rho_v$, and the expression $\beta = 3.4e^{-0.1(h_w/t_w)} + 1.1e^{-4.8\rho_v}$ was developed as an expression for this relationship, as shown in Figures 92 and 93. Note that since the $R^2$ value was equal for each of the individual relationships between $\beta$ and $h_w/t_w$ (i.e. $6.8e^{-0.1(h_w/t_w)}$) and $\beta$ and $\rho_v$ (i.e. $2.2e^{-4.8\rho_v}$), so each contribution was given an equal 50/50 weighting. With regards to the $\delta$ hysteretic parameter, the only DSRMSW design parameter with significant effect was the vertical reinforcement ratio, $\rho_v$, and the expression for this relationship is shown in Figure 94 (i.e. $1.4\rho_v - 0.04$). Therefore, while the values for $\beta$ and $\delta$ hysteretic parameters will vary for DSRMSWs based on their $h_w/t_w$ ratio and $\rho_v$, the $\alpha$, $\gamma$, $\kappa$, and $\eta$ parameters values may be assumed be constant at those shown in Table 16.

Table 16. Recommended hysteretic parameters for vertical and horizontal springs.

<table>
<thead>
<tr>
<th>ID</th>
<th>$\alpha$</th>
<th>$\beta$</th>
<th>$\delta$</th>
<th>$\gamma$</th>
<th>$\eta$</th>
<th>$\kappa$</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1</td>
<td>1.50</td>
<td>0.65</td>
<td>0.60</td>
<td>1.16</td>
<td>0.15</td>
<td>0.20</td>
</tr>
<tr>
<td>W2</td>
<td>1.30</td>
<td>0.35</td>
<td>0.40</td>
<td>1.16</td>
<td>0.15</td>
<td>0.20</td>
</tr>
<tr>
<td>W3</td>
<td>1.25</td>
<td>0.30</td>
<td>0.30</td>
<td>1.15</td>
<td>0.13</td>
<td>0.20</td>
</tr>
<tr>
<td>W4</td>
<td>1.30</td>
<td>1.00</td>
<td>0.15</td>
<td>1.16</td>
<td>0.15</td>
<td>0.20</td>
</tr>
<tr>
<td>W5</td>
<td>1.30</td>
<td>0.35</td>
<td>0.25</td>
<td>1.16</td>
<td>0.15</td>
<td>0.20</td>
</tr>
<tr>
<td>W6</td>
<td>1.30</td>
<td>0.65</td>
<td>0.40</td>
<td>1.16</td>
<td>0.15</td>
<td>0.20</td>
</tr>
<tr>
<td>W7</td>
<td>1.30</td>
<td>0.65</td>
<td>0.40</td>
<td>1.16</td>
<td>0.15</td>
<td>0.20</td>
</tr>
<tr>
<td>W8</td>
<td>1.20</td>
<td>0.80</td>
<td>0.30</td>
<td>1.15</td>
<td>0.15</td>
<td>0.20</td>
</tr>
<tr>
<td>MEAN</td>
<td>1.31</td>
<td>0.59</td>
<td>0.35</td>
<td>1.16</td>
<td>0.15</td>
<td>0.20</td>
</tr>
<tr>
<td>STD</td>
<td>0.08</td>
<td>0.23</td>
<td>0.13</td>
<td>0.00</td>
<td>0.01</td>
<td>0.00</td>
</tr>
<tr>
<td>COV</td>
<td>6.2%</td>
<td>38.7%</td>
<td>35.7%</td>
<td>0.4%</td>
<td>5.6%</td>
<td>0.0%</td>
</tr>
</tbody>
</table>
Figure 96. Relationship between $\beta$ and $h/w$.

Figure 97. Relationship between $\beta$ and $\rho_v$. 

$\beta_{h/t} = 6.8e^{-0.1(h/t)}$

$R^2 = 0.66$

$\beta_{\rho_v} = 2.2e^{-4.8\rho_v}$

$R^2 = 0.66$
Figure 98. Relationship between $\delta$ and $\rho_v$.
6.3.2 Macroscopic-Level Sensitivity Analysis

This section of the chapter describes the sensitivity analyses conducted on the level of element and spring discretization of MVLEMs. Both of these discretization types have significant effects on the modelling effort, computational time, and accuracy of MVLEMs. These sensitivity analyses also use MVLEMs simulating the experimental testing of the eight full-scale DSRMSW test specimens from the experimental phase of the study for verification. All of the test specimens were individually modelled using three separate general levels of discretization: “high,” “medium,” and “low.” Figures 95 to 99 diagrammatically depict the three levels of element discretization for each of the test specimens, whereas Figure 100 to 104 depict the three levels of spring discretization. For the sensitivity analyses investigating the level of element discretization, the MVLE spring discretization followed the “medium” level shown in Figure 95c to 99c. For the parametric analyses studying the level of spring discretization, the MVLE element discretization followed the “medium” level shown in Figure 100c to 104c. This section provides general recommended element and spring discretization levels for DSRMSW MVLEMs.
Figure 99. Specimens W1, W2, and W5: (a) physical wall, (b) high element discretization, (c) moderate element discretization, and (d) low element discretization.

Figure 100. Specimen W3: (a) physical wall, (b) high element discretization, (c) moderate element discretization, and (d) low element discretization.

Figure 101. Specimen W4: (a) physical wall, (b) high element discretization, (c) moderate element discretization, and (d) low element discretization.

Figure 102. Specimens W6 and W7: (a) physical wall, (b) high element discretization, (c) moderate element discretization, and (d) low element discretization.

Figure 103. Specimen W8: (a) physical wall, (b) high element discretization, (c) moderate element discretization, and (d) low element discretization.
Figure 100. Specimens W1, W2, and W5: (a) physical wall, (b) high spring discretization, (c) moderate spring discretization, and (d) low spring discretization.

Figure 101. Specimen W3: (a) physical wall, (b) high spring discretization, (c) moderate spring discretization, and (d) low spring discretization.

Figure 102. Specimens W6 and W7: (a) physical wall, (b) high spring discretization, (c) moderate spring discretization, and (d) low spring discretization.

Figure 104. Specimen W8: (a) physical wall, (b) high spring discretization, (c) moderate spring discretization, and (d) low spring discretization.

Figure 105. Specimen W4: (a) physical wall, (b) high spring discretization, (c) moderate spring discretization, and (d) low spring discretization.
6.3.2.1 Element Discretization

The three levels of element discretization shown in Figure 84 to Figure 88 were defined based on observations from the experimental phase of this study. The “high” level of element discretization model incorporates two vertically-orientated MVLEs at each of the wall’s horizontal mortar joints and two vertically orientated MVLEs over the height of each course of grouted concrete block masonry units, as shown in Figure 95b to Figure 99b. Most of the horizontal mortar joint MVLEs are defined with a height of 1.5 times half of the 10 mm thickness of the physical joints to account for the effects of strain penetration beyond the edges of the mortar joints. The base mortar joint is however represented by a MVLE with a 50 mm height due to the additional strain penetration observed in this location during the experimental tests. The “medium” level of element discretization model incorporates only one vertically-orientated MVLEM for each horizontal mortar joint and one MVLEM for each masonry course. Similar to the “high” level of element discretization, the horizontal mortar joint MVLE heights were typically 15 mm except for a special 50 mm-high MVLE employed for the base mortar joint. The “low” level of element discretization model incorporates a single MVLE type over the combined height of three masonry courses and three horizontal mortar joints, which roughly represents half of the plastic hinge height for most of the walls. This is based on the recommendations on plastic hinge length provided by Brzev and Anderson [36] as well as the experimental observations from the experimental phase specimen tests. Unlike for the “high” and “medium” element discretization levels, where different material factors were incorporated for the mortar and grouted masonry MVLEs, the “low” element discretization level used MVLEs with a smeared masonry strength.

As shown in Figure 109 to Figure 112, the “low” level of element discretization generally results in conservative predictions of the modelled wall’s lateral load relative to the experimental
data at lower drift levels (e.g. Figure 111b and Figure 112a). Furthermore, the “low” element discretization tends to be less capable of capturing the strength degradation portion of the lateral load hystereses compared to the “medium” and “high” levels of element discretization (e.g. Figure 109b and Figure 110a). This may be due to the inability of the first element to capture the strain localization effects that were typically observed to occur when toe-crushing initiates locally near the base of the wall toes. The differences between the “medium” and “high” levels of element discretization tend to be relatively minor, despite the significant savings in modelling effort and computational time provided by the “medium” level of element discretization, leading to the conclusion that the “medium” level of element discretization may be the preferred option of the three discretization levels investigated.

![Figure 109. Element-discretization sensitivity study summary for specimens W1 (a) and W2 (b).](image)
Figure 110. Element-discretization sensitivity study summary for specimens W3 (a) and W4 (b).

Figure 111. Element-discretization sensitivity study summary for specimens W5 (a) and W6 (b).
6.3.2.2 Spring Discretization

The three levels of spring discretization shown in Figure 89 to 93 were defined based on the cross-sectional components of the experimental test from the experimental phase of this study. The vertical springs were generally discretized based on segments of the wall cross-section surrounding the vertical reinforcing bars. Therefore, as shown in Figure 89 to 93, the vertical spring wall segments were more heavily concentrated near the wall end zones, and more sparsely spread over the interior of the wall length. Near the wall end zones, the “high” level of spring discretization model divides each transverse masonry face shell into longitudinally-orientated two vertical springs, each longitudinal face shell three longitudinally-orientated vertical springs, and each grouted concrete masonry unit cell into three longitudinally-orientated vertical springs. Near mid-length of the wall, the spring segment areas increase by a factor of approximately 3.0 as these portions of the wall were observed not to experience significant nonlinear behaviour. When
compared to the “high” spring discretization level, the wall segments at the “medium” spring discretization level segments are roughly double the area for the transverse face shells, triple for the longitudinal face shells, and triple as well for the grouted cells. Both the “high” and “medium” spring discretization levels incorporated the individual material properties for the masonry units, grout, mortar, and steel reinforcement components of the spring wall segments. For the “low” spring discretization level, the material properties for the masonry units, grout, and mortar were transformed into a smeared masonry material that was applied over the significantly larger spring wall segments shown in Figure 89d to 93d.

As shown in Figure 113 to Figure 116, based on the investigated experimental tests, MVLEs employing the “low” level of spring discretization generally have difficulty in capturing toe-crushing and strength degradation (e.g. Figure 113a and Figure 115a). This is likely due to the relatively unrefined spring discretization at the ends of the wall failing to capture the initiation of toe-crushing at the outer transverse face shells, which precipitates toe-crushing in the adjacent grouted cores and longitudinal face shells. Conversely, the “high” level of spring discretization is generally capable of capturing strength degradation following the initiation of toe-crushing; however it was found to occasionally encounter convergence problems at even relatively low levels of drift (e.g. Figure 114a and Figure 116a) and required significantly more modelling effort and computation time. The “medium” level of spring discretization was found to produce the preferred balance between the ability to capture strength degradation, while still maintaining relatively robust convergence characteristics, and keeping modelling effort and computational time comparatively low.
Figure 113. Spring-discretization sensitivity study summary for specimens W1 (a) and W2 (b).

Figure 114. Spring-discretization sensitivity study summary for specimens W3 (a) and W4 (b).
Figure 115. Spring-discretization sensitivity study summary for specimens W5 (a) and W6 (b).

Figure 116. Spring-discretization sensitivity study summary for specimens W7 (a) and W8 (b).
6.4 Model Validation

Following the parametric and sensitivity analyses carried out in the previous sections of the chapter, this section describes the modelling validation. Using only the recommended mean hysteretic parameters identified in the microscopic-level parametric analyses and the “medium” levels of element and spring discretization described in the macroscopic-level sensitivity analysis, MVLEMs are generated to model the experimental results of three DSRMSW specimens tested by Shedid [94]: specimens “Wall 2,” “Wall 4,” and “Wall 5.” (the other specimens in the test series were affected by partial grouting issues during construction). The comparison between the numerically-predicted MVLEM load-displacement responses and the corresponding experimental results of the test specimens are shown in Figure 117 to Figure 119. The specimen data presented in each of these figures by Shedid describe the main attributes of each wall [94]. As such, it is worth noting that all of these specimens are rectangular, were not subjected to superimposed axial load demands, and only specimen Wall 5 has a vertical reinforcement ratio that is typically used in practice, whereas specimens Wall 2 and Wall 4 incorporate higher-than-typical reinforcement ratios.

It can be seen that the predicted MVLEM responses are in generally good agreement with the experimental results, particularly for the lighter-reinforced specimen, Wall 5, which is more closely related to the test specimens from the experimental phase of the current study in terms of vertical reinforcement ratio. For specimen Wall 5, the initial stiffness, yield strength, ultimate strength, drift capacity, and strength degradation are well captured and generally approximately within 10% of the experimental results. The unloading curve is also relatively well captured but to a somewhat lesser degree. The agreement between the predicted MVLEM responses and the experimental results for specimens Wall 2 and Wall 4 are still relatively good in terms of initial
stiffness, yield strength, ultimate strength, and drift capacity, but strength degradation is in weaker agreement. The MVLEM predicts more rapid strength degradation than the experimental results at lower drift levels (less than 1.7% drift), but then underestimated the rate of strength degradation at higher drift levels (greater than 1.7% drift). Even more apparent is the discrepancy between the predicted and experimental results for the unloading curves, which the MVLEM significantly underpredicts the initial unloading slope.

Considering this brief numerical validation study, it is recommended that the proposed hysteretic parameters and levels of element and spring discretization discussed in the previous sections of the chapter be used for DSRMSWs with typical reinforcement ratios. Special DSRMSWs with higher reinforcement ratios may need additional study to develop more suitable hysteretic parameters and element and spring discretization levels.

![Figure 117. MVLEM validation test based on the lateral load-displacement response of Wall 2.](image)
Figure 118. MVLEM validation test based on the lateral load-displacement response of Wall 4.

Figure 119. MVLEM validation test based on the lateral load-displacement response of Wall 5.
6.5 Summary and Conclusions

This chapter provided parametric and sensitivity analyses at the microscopic and macroscopic modelling levels, respectively, for the first use of an MVLEM approach to predict the response of DSRMSWs. These analyses were carried out using the experimental results from the set of eight full-scale DSRMSW specimens tested in the experimental phase of this study. The results of the analyses provided a set of recommended hysteretic parameters and proposed element and spring discretization levels. Since DSRMSWs have few geometrical design parameters, i.e. typically one standard-sized masonry unit placed in a standard-sized mortar joint with single-layer vertical and horizontal reinforcement, the “medium” level discretization level may be utilized for general DSRMSW applications. The recommended hysteretic parameters and proposed element and spring discretization levels were then tested in a numerical validation to predict the response of an additional set of three full-scale DSRMSW specimens experimentally tested by Shedid [94]. The numerical validation results indicated that the proposed modelling approach provides the better results for DSRMSWs with lighter vertical reinforcement ratios, which are more common in practice, but can still provide reasonable predictions for the initial stiffness, yield strength, ultimate strength, and drift capacity of more heavily reinforced walls. As these parameters are of the most interest for determining the displacement-based performance of walls, the proposed MVLEM approach may provide a good candidate for designers wishing to determine the PBD capacities of DSRMSWs. Overall, the MVLEM approach provides a model that is simple to understand conceptually, robust for typical DSRMSW applications, carries a relatively low level of modelling effort, is capable of capturing shear deformations, and has good accuracy for modest computational time, which few other modelling approaches provide. However, while the MVLEM hysteretic parameters developed based on the current test series have been demonstrated to produce generally
good results for similar walls tested by other researchers, further investigation and calibration is required to ensure the model performs well for DSRMSWs with substantially different design parameters compared to the current test series.
Chapter 7: Conclusions

7.1 Introduction

This study has investigated the performance of DSRMSWs subjected to in-plane seismic loads, with a special focus on the out-of-plane response, which has been typically neglected in past experimental studies on DSRMSWs. This study included both experimental and analytical phases to achieve the goals listed in Section 1.2, which provided a series of conclusions on the in-plane and out-of-plane response of DSRMSWs loaded in-plane.

7.2 Experimental Phase

The DSRMSW specimens tested in the experimental phase of this study demonstrated good performance in terms of ductility and energy dissipation during the simulated in-plane seismic loading to which they were subjected. The $\mu_u$ factor determined from the ductility-based approach employed in this phase was found to have a mean value of 8.7, which is substantially higher than the very similar ductility-related force modification factor, $R_d$, value of 3.0 from CSA S304-14 [17] for DSRMSWs. However, the 10$^{th}$ percentile $\mu_u$ factor from the tested specimens correlated well to the CSA S304-14 [17] value, helping to substantiate this figure.

The in-plane responses of the specimens were generally characterized by initial ductile flexural failure modes that would typically eventually transition to a toe-crushing failure mode that would occur near the attainment of the ultimate lateral load capacity of the wall. Subsequently, other failure modes, such as sliding, rocking, bar-fracture, shear-flexure, and lateral instability, would then often occur following the attainment of the ultimate capacity. Lateral instability was determined to be the primary failure mode related to the out-of-plane response of the specimens. The out-of-plane response of the specimens was generally characterized by peak out-of-plane
displacements that would increase with increasing in-plane lateral displacement demands. The out-of-plane displacements would typically be highest during mid-cycle of the imposed drift amplitudes from the lateral loading protocols, and smallest when the end zone was either in maximum tension or maximum compression at the beginning and end of the drift cycles. Despite all of the specimens being designed with slenderness limits exceeding the CSA S304-14 [17] $h_u/t_w$ limits by 32% to 79%, and two of the specimens exhibiting significant out-of-plane displacements that surpassed 35% of the wall thickness, only one specimen failed due to global lateral instability. Even this single global lateral instability was only a Class B instability, which occurred after the ultimate lateral load capacity of the wall had been reached. Since current masonry design codes employ a force-based approach for the design of DSRMSWs, this appears to suggest that current slenderness limits, typically in the form of $h_u/t_w$ limits, may be overly conservative or potentially inappropriate when used alone to assess the lateral stability of DSRMSWs. This conclusion is reinforced by a number of factors that were found to affect the out-of-plane response of DSRMSWs and which cannot be adequately accounted for by $h_u/t_w$ limits alone.

### 7.3 Analytical Phase

The analytical phase used the observations from the experimental testing in conjunction with the numerical test data from the experimental phase for several purposes: a) to classify each of the prominent flexure-dominant failure modes affecting DSRMSWs and how they interact with one another; b) to compare current leading masonry design codes to determine the most effective methodologies for developing force-based codes provisions; and c) to calibrate and verify an MVLEM specialized for DSRMSWs, which can be efficiently and reliably used by practicing design engineers that are following a PBD approach for the analysis of DSRMSWs.
The failure mode classifications and detailed summaries provide greater insight into the development of these failure modes and the circumstances in which they are most likely to occur. These circumstances range from the effects of different wall geometries, anticipated drift levels for their occurrence, and the influence of failure modes that have already initiated before their occurrence. The failure mode summaries provide practicing design engineers a greater understanding of how the walls that they design may behave under in-plane seismic loading.

Comparison of the current masonry design codes of Canada (CSA S304-14 [17]), USA (TMS 402/602-16 [67]), New Zealand (NZS 4230:2004 [68]), and EU (Eurocodes 6 and 8 – BS EN 1996-1-1:2005/BS EN 1998-1:2004 [69], [70]) revealed a number of significant similarities and differences among the methodologies behind the code provisions. The provisions for shear-flexure were found to have the best agreement among the codes in terms of explicit code provisions due to very similar methodologies, whereas the provisions for sliding shear were found to be the most dissimilar among the codes primarily due to hangovers from allowable stress design still present in some codes. Despite beam theory being used by all codes for determining the ductile-flexure capacity of DSRMSWs, some codes prescribe limits on the compression zone length, neglect the effect of compression reinforcement, and restrict tensile strain in the vertical reinforcement, which can lead to high degrees of conservatism in their lateral load capacity predictions. The displacement code predictions differed significantly between the codes and were found to be highly sensitive to the code-prescribed plastic hinge lengths, which varied by as high as a factor of 3. Limits on the compression strain in the masonry as well as the length of the compression zone and the tensile strain in the vertical reinforcement also had a considerable impact on the predictions. As noted earlier, based on the small number of specimens in this experimental study and the fact that no Class C global lateral instability was observed during the testing despite
efforts to force its occurrence, the codes that did not prescribe \( h_\text{w}/t_\text{w} \) restrictions may be more appropriate than those that do. Overall, due to the differences in the different code predictions, it appears that masonry design code committees could benefit from additional code comparison studies to determine how they may improve future provisions.

The detailed numerical data from the experimental phase was first used in parametric analyses to calibrate the microscopic-level hysteretic parameters of the vertical and horizontal springs for MVLEMs representing DSRMSWs for the first time. Following the microscopic-level parametric analyses and calibration, macroscopic sensitivity analyses were carried out to determine the level of spring and element discretization for the optimal balance of computational expense and model accuracy. The proposed calibrated model was found to predict reasonable accuracy not only the lateral load-displacement envelope responses of not only the eight specimens from the experimental phase but also three other DSRMSWs tested by others with very low computational expense and a good degree of accuracy. The MVLE modelling methodology presented may be used by practicing engineers as a guideline when developing their own models to estimate the PBD capacities of DSRMSWs.

7.4 Contributions of the Thesis

The study documented in this dissertation has made contributions to the body of knowledge relating to the seismic performance of DSRMSWs by completing the objectives of Section 1.2:

1. By conducting the experimental testing described in this dissertation, the out-of-plane response of full-scale DSRMSWs loaded in-plane was captured in more detail than had been accomplished in any previous study. Several of the wall specimens also had \( h_\text{w}/t_\text{w} \) ratios that exceeded all previous studies on DSRMSWs, thus representing the best
candidates at assessing the current $h_u/t_w$ limits proposed by CSA S304-14 [17] and other masonry design codes.

2. Analysis of the experimental findings provided important numerical data and observations on the in-plane and out-of-plane performance of DSRMSWs and support the appropriateness of the ductility-related modification factor used by CSA S304-14 [17] and NBCC 2015 [18] for this wall type.

3. Key parameters influencing the out-of-plane response of DSRMSWs subjected to in-plane seismic effects were identified have not only led to the conclusion that $h_u/t_w$, when used alone, may not be appropriate for assessing the lateral stability of DSRMSWs, but also provide a basis for future research on the topic. While there are fundamental differences between DSRMSWs and RCSWs, many of the findings from this experimental study may prove useful during ongoing and future research on the lateral stability of RCSWs.

4. A suite of failure mode summaries was developed for reference by practicing design engineers designing DSRMSWs to improve the state of knowledge regarding the behaviour of DSRMSWs under seismic loading, which is particularly useful when employing capacity design principles.

5. The failure mode summaries were used in a comparison study of current masonry design codes for each of the prominent failure modes affecting DSRMSWs subjected to simulated in-plane seismic loading. This code comparison provided important recommendations for future code provisions based on the accuracy of the code predictions relative to the test results from the experimental phase of this study.

6. An analytical model was calibrated and verified using the test data from the experimental phase of the study and validated against a test series from another DSRMSW experimental
program by others. The model was demonstrated to accurately simulate the lateral load-displacement envelope response of DSRMSWs at a computational cost that is low enough for use by practicing design engineers.

### 7.5 Recommendations for Future Research

The need for further research in specific areas has been discussed at various points in this thesis, particularly in Chapters 4 and 6, where additional testing and calibration is needed to confirm the results of necessarily relatively small experimental test programs. The most pertinent needs are briefly listed below:

- Following the current experimental program, the diameter of the vertical reinforcing bars is believed to potentially have critical effects on the out-of-plane response of DSRMSWs that are subjected to in-plane loading. This is especially important for DSRMSWs that have already experienced toe-crushing, thus most related to Class A and B instabilities. As discussed in Chapter 4, if the bar diameter is relatively small (less than 20 mm), a local Class A instability is expected following toe-crushing. However, if the bar diameter is large (greater than 25 mm), a global Class B instability may be expected. As this experimental study exclusively used 15 mm-diameter vertical reinforcing bars in the wall end zones, further experimental testing studying the effect of this parameter is recommended for future amendments to lateral instability provisions in masonry design codes.

- The combined effect of in-plane and out-of-plane displacements occurring simultaneously may potentially lead to a higher propensity for a lateral instability failure mode. This type of loading was not investigated in the experimental study presented in this thesis and should thus be considered for future experimental studies on DSRMSWs.
• The specimens tested in this experimental study used specimens with shear reinforcement that was in excess of the limits provided by CSA S304-14 [17], and this significantly helped control shear cracking. Lower levels of shear reinforcement are expected to increase the degree of shear cracking experienced in DSRMSWs and thus also likely decreasing the propensity of a lateral instability failure mode. Future experimental studies on DSRMSWs should consider using levels of shear reinforcement that are closer to the minimum code requirements to study this effect.

• While most of the test specimens in this research were based on short structures with relatively low axial loads, future experimental research should also study the effect of higher axial loads on the ductility and vulnerability to lateral instability of the DSRMSWs.

• While the experimental study presented in this thesis did not find h_u/t_w limits to be particularly relevant when acting alone as the sole criterion for code provisions assessing the lateral instability of DSRMSWs, significantly more testing on DSRMSW specimens with h_u/t_w values significantly exceeding the code limits is needed to confirm these findings.

• Only a relatively small series of test specimens was used to compare the effectiveness of the four leading masonry design codes investigated in Chapter 5. Significantly more experimental tests should be used in further comparisons to confirm the findings from this portion of the analytical study.

• Further calibration of the MVLE modelling approach proposed in Chapter 6 is required with experimental data from additional experimental test specimens to fully confirm the hysteretic spring parameters determined by the parametric analyses. Moreover, with additional test data, the development of relations between these hysteretic parameters and
physical parameters of individual DSRMSWs may be possible and allow the model to be applied more generally for DSRMSWs with design parameters that are substantially different from those used in the test series from this research. Additionally, if more experimental data was to become available for the relationships between the combined effects of global overturning moments and sliding displacements on the bar-fracture capacity of vertical reinforcement, this information could be used to refine the hysteretic force-displacement relations for the vertical springs such that the rocking failure mode could be better captured by the MVLEM. Similarly, if more experimental data were produced from DSRMSWs exhibiting large out-of-plane displacements or lateral instability, the vertical spring hysteretic force-displacement relations could be modified to capture their effect on the lateral load-displacement response of DSRMSWs.

- If future experimental programs on DSRMSWs can produce several specimens that fail due to Class C or B instability, this data should also be used to calibrate the hysteretic spring parameters such that they may account for the effects of out-of-plane displacements on the in-plane lateral load-displacement performance of DSRMSWs.
Bibliography


In-Plane Reversed-Cyclic Loads.” Proceedings of the 15th World Conference on Earthquake Engineering, Lisbon, Portugal.


Appendices

Appendix A : Additional Test Photos
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