Out-of-Plane Stability of Reinforced Masonry Shear Walls under Seismic Loading: In-Plane Reversed Cyclic Testing

by

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Abstract

Reinforced concrete block masonry shear walls (RMSWs) often constitute the principal seismic force resisting system in masonry structures in Canada. During an earthquake, these walls experience the combined effects of axial gravity loading and overturning moments due to in-plane lateral seismic forces. This loading precipitates out-of-plane instability when the longitudinal reinforcement in the wall end zones is subjected to cycles of high tensile strain followed by compression. The Canadian masonry design standard (CSA S304.1-04) [Canadian Standard Association 2004] stipulates stringent height-to-thickness ratio limits for the seismic design of ductile RMSWs. Experimental research and earthquake evidence have demonstrated this failure mechanism in reinforced concrete shear walls loaded in-plane. However, similar evidence of the mechanism occurring in RMSWs is not available. This provided motivation for the research study described in this thesis.

The research presented here represents the second phase of a comprehensive multi-phase research program. The first phase involved the experimental testing of full-scale reinforced masonry (RM) column-like specimens subjected to uniaxial cyclic tension-compression loading. The testing provided valuable insight into the out-of-plane instability as it occurs in RM. The second phase of the research program focused primarily on the lateral reversed-cyclic experimental testing of two full-scale, slender RMSWs with height-to-thickness ratios of 27, well exceeding the CSA S304.2 limits. The target failure mode was an out-of-plane failure mechanism. The results contribute unique benchmark data for the qualitative and quantitative assessment of the factors influencing out-of-plane instability of RMSWs as well providing better understanding of the mechanism itself. The effect of applied axial stress on out-of-plane instability is evaluated and possible other influential design parameters are discussed.

From the results of the experimental study, it was concluded that the applied axial stress is a critical factor in the initiation of out-of-plane instability. This factor had effects on many other parameters, the most important of which was the development of tensile strain in the vertical
reinforcement as well as the width and distribution of cracks over the plastic hinge height. These results indicate that the height-to-thickness ratio alone may not be an adequate factor governing the occurrence of out-of-plane instability in RMSWs.
Preface

A small portion of Chapter 3.3 on the Test Setup was described briefly in a published paper by Azimikor, N., [Brook Robazza], Elwood, K. J., Anderson, D. L., and Brzev, S. (2012). An Experimental Study on the Out-of-Plane Stability of Reinforced Masonry Shear Walls Under In-Plane Reversed Cyclic Loads, World Conference on Earthquake Engineering (15WCEE), Lisbon, Portugal. I wrote the section of the paper included in this thesis.

Section 3.2.3 on Specimen Material Properties was based on work conducted in the Structural Laboratory of the British Columbia Institute of Technology (BCIT), the testing facilities at Basalite, Vancouver, and the Structures Laboratory of the University of British Columbia (UBC). The bulk of the testing was performed by BCIT civil engineering students Wen Cheng Yang, Julien Levasseur and Eric Wan under the supervision of the main laboratory technician, Ken Zeleschuk. I constructed nearly all of the materials test specimens (concrete and grout cylinders, rebar coupons, concrete block prisms, grout prisms, mortar cubes, and mortar flow and slump tests) myself but assisted in only a small number of the actual materials tests. I was responsible for the final post-processing of the test data and reporting on the testing procedure.

A large portion of Section 4.2 on the Experimental Observations of W1 was included in a published paper by [Brook Robazza], Elwood, K. J., Anderson, D. L., and Brzev, S. (2012) In-Plane Seismic Behaviour of Slender Reinforced Masonry Shear Walls: Experimental Results. 12th Canadian Masonry Symposium (CMS), Vancouver, B.C., Canada. I designed and conducted the testing which the paper described. I also wrote the manuscript with the assistance from the other three authors.
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Dedications

To my beloved family, Ray, Kymn, and Rance as well as my loving wife, Angelique
CHAPTER 1

Introduction

Most new loadbearing masonry walls in Canada consists of hollow concrete block units laid in a mortar bed and reinforced with vertical and horizontal steel reinforcing bars running through the block cells and along the mortar bed joints. This type of masonry construction is termed reinforced masonry (RM) as opposed to veneer and infill masonry. Shear walls are one of the main forms of RM construction. Reinforced masonry shear walls (RMSWs) act as the primary structural element in carrying the loads induced upon the wall and transferring them to the foundation. These loads may consist of lateral in-plane loads due to wind or seismic forces in addition to the more permanent vertical gravity loads.

In the earthquake prone regions of Canada, the seismic performance of a building often provides the governing design criteria for a structure. Naturally then the choice of building material is frequently heavily influenced by its seismic performance. Numerous experimental studies and evidence from past earthquakes have shown that RMSWs are capable of yielding a high degree of seismic resistance while still providing economical solutions for the seismic design of low- and mid-rise buildings.

The seismic resistance of Canadian RM buildings is determined from the lateral load capacity of the RMSWs according to the Canadian Masonry Design Standard (CSA S304.1-04). The classification of RMSWs by CSA S304.1-04 (Canadian Standard Association, 2004) and the National Building Code of Canada (NBCC) 2010 depends on their ability to dissipate the energy...
imparted upon them during a seismic event. Currently there are two wall classes which differ in
the ductility level and the corresponding seismic performance: “Conventional Masonry Shear
Walls” with a ductility factor of $R_d = 1.5$ and “Moderately Ductile Shear Walls” with a ductility
factor of $R_d = 2.0$. The latter class ($R_d = 2.0$) requires more complex detailing to ensure better
performance and ductility during an earthquake.

Some of the CSA S304.1-04 design requirements are very stringent and have made RMSWs
impractical for many common design applications in Canada. One of the most prohibitive
requirements imposed upon “Moderately Ductile Shear Walls” is with regard to their height-to-
thickness (h/t) ratios. CSA S304.1-04 restricts RMSWs to h/t ratios of 14 to 20 depending on the
wall class. “Limited Ductility Shear Walls” ($R_d = 1.5$) are limited to an h/t ratio of 18,
“Moderately Ductile Shear Walls” ($R_d = 2.0$) have a maximum h/t ratio of 14, and “Moderately
Ductile Squat Shear Walls” also with an $R_d = 2.0$ are restricted to an h/t ratio of less than 20.
Since it is most prevalent and practical to design RM buildings with RMSWs composed of a
single wythe of 190 mm standard concrete block units, this effectively imposes a height
restriction of as low as 2.7 m for such walls. This inherently severely limits the construction
economy and practicality of the use of RMSWs in many design applications. Additionally, since
2005, high-importance (post-disaster) buildings designed in accordance with NBCC must be
detailed an $R_d$ factor of 2.0 or higher; this places the most inhibitive restrictions on structures
such as fire halls, police stations, and even pump stations which formerly used to be commonly
built using RMSWs. As this requirement on post-disaster buildings in uniform across Canada
and irrespective of the level of seismic hazard, RMSWs in urban centres with low seismic risk
such as Toronto, Winnipeg, and Calgary much comply to these regulations. As a result, since
2005 use of typical RMSWs in the design and construction of high-importance buildings in
Canada has been limited.

RMSWs experiencing in-plane seismic loading are subjected to the combined effects of lateral
shear forces, axial loads, and overturning bending moments. Factors which must be considered
include the level of axial compressive stress, the amount of horizontal and vertical reinforcement,
the wall aspect ratio, and the mechanical properties of the masonry and steel reinforcement.
When these loads act together in the wall plane, as they do in a seismic event, one of the possible
flexural failure mechanisms which may occur is out-of-plane instability. Of particular
importance for this mechanism are the combined effects of axial load and in-plane bending moments. The h/t ratio limits prescribed by CSA S304.1-04 are intended to prevent the possibility of out-of-plane instability failure in RMSWs due to in-plane seismic loading. Figure 1-1 illustrates out-of-plane instability due to the effects of in-plane loading.

Anderson and Brzev in their Seismic Design Guide for Masonry Buildings (2009) give a substantial explanation of this failure mechanism based on the findings of Paulay (1986) and Paulay and Priestly (1992, 1993). When a RMSW experiences considerable curvature ductility demand, large tensile strains develop in the vertical reinforcement at the tension end zone in the bottom portion of the wall. As this occurs, uniformly spaced flexural cracks of significant width begin to develop over the plastic hinge length. During the subsequent unloading cycle, the strains in the reinforcement reverse into compression. At this stage, the compression stresses in the wall are resisted solely by the vertical reinforcing bars, which may start to displace laterally due to their limited lateral stiffness. From this point, two mechanisms of response with different
consequences may follow. The first mechanism develops if the flexural cracks close and the masonry in the end zone that was previously subjected to tension begins to resist compression; in this case, lateral stiffness of the compression region of the wall is restored and the wall remains straight over its height. The second mechanism develops if the flexural cracks do not close before the lateral displacement reaches a critical value; in that case, the out-of-plane displacements may continue to increase, and out-of-plane instability may occur at the wall end zone. Figure 1-2 illustrates the two potential mechanisms associated with out-of-plane instability.

Despite the fact that the strict h/t ratios are based upon this out-of-plane instability failure mechanism, there is very little experimental evidence supporting this mandate. A comprehensive literature review performed by Azimikor et al. (2012) revealed an absence of experimental evidence related to out-of-plane instability in ductile RMSWs. Reinforced concrete walls have demonstrated out-of-plane instability failure mechanisms due to the combined effects of axial loads and in-plane bending moments in experimental studies but there is no indication that RM walls are also susceptible to this failure mechanism. In addition to the lack of experimental evidence, a review of the seismic design provisions of international masonry design standards revealed that the limits on RMSW h/t ratios are much less stringent than those provided by the CSA S304.1-04 provisions and in some cases are even non-existent. These factors prompted the need for a research program which would characterize out-of-plane instability in RMSWs and develop rational criteria for out-of-plane instability in these walls.
Figure 1-2 Out-of-plane instability mechanism in a RMSW, adapted from Paulay and Priestley (1992): a) wall in unstressed state, b) wall after inelastic tensile excursion, c) large lateral displacement (flexural cracks do not close), d) small lateral displacement (flexural cracks close), e) strains at section A-A, f) strains at section B-B, g) strains at section C-C
1.2 Objective and Scope

The overall objective of this research is to ensure safe and economical seismic design of RMSW structures in Canada by establishing rational criteria for assessing the out-of-plane stability of RMSWs subjected to in-plane seismic loading. The specific objectives are:

i. To develop a rational and practical analysis procedure for assessing the out-of-plane stability of RMSWs using criteria related to practical wall design parameters, and

ii. Evaluate the appropriateness of a height-to-thickness (h/t) ratio limit as a prescriptive design requirement for the seismic design provisions of the Canadian Masonry Design Standard CSA S304.1.

The results of this experimental research program will assist in drawing a more accurate understanding of how or if the slenderness of RMSWs is related to their ductility capacity. The study will also examine other design parameters that influence the out-of-plane stability of RMSWs.

This thesis has been undertaken as a part of a four-year, two-phase research program which started in November 2010. Phase 1 of this study was based on investigating the out-of-plane stability RMSW end zones by subjecting reinforced masonry columns to cycles of axial reversed cyclic loading. The current research study is a part of Phase 2 of the program, and the focus is on examining the factors contributing to the out-of-plane instability of RMSWs. This phase involves the testing of full-scale wall specimens subjected to in-plane cyclic lateral loads and bending moments applied atop the wall specimens that simulate seismic demands. In total, two 140 mm thick wall specimens have been tested. The specimens are representative of RMSWs found in Canadian RM buildings, although with larger h/t ratio (close to 27). The first specimen represented the bottom portion of a relatively tall wall typical of a multi-storey building, which is subjected to combined effect of lateral shear force and overturning moment. The second specimen represented a wall in a single-storey, warehouse type building subjected to lateral shear force at the top of the wall. The design of these wall specimens took into account the findings of Phase 1 study in order to attempt to force an out-of-plane instability failure mechanism. This
thesis describes the tests on the first two rectangular wall specimens. Note that the Phase 2 research program will also include additional rectangular wall specimens, as well as T-shaped wall specimens; the specimens will be composed of the thicker, more common 190 mm thick concrete block units.

The results of this research program are expected to contribute to the CSA S304.1 standard for the design of masonry structures in Canada which is currently in the review stage and is scheduled for release in 2014. From this investigation it is possible to evaluate the current CSA S304.1 h/t ratio limitations. It is expected that the findings of this research will demonstrate that the current h/t ratio restriction prescribed by CSA S304.1-04 is excessively stringent. The results of this project have a potential to make a significant impact upon seismic design and design applications of RMSWs in Canada, and also provide useful information for national and international academic community interested in the seismic response of masonry structures.

1.3 Thesis Organization

This section outlines the content of the thesis with a brief description of each chapter. The chapter summaries are listed in the order of which they appear in the thesis. The thesis will describe and discuss the following: previous related experimental research studies, the wall specimen design and construction, the experimental test setup, the testing procedure, the wall specimen response and the associated experimental observations followed by comparison of the experimental results between the two tests, a description of an analytical model designed to simulate the behaviour of the compression zone of a RMSW and how axial loading affects the out-of-plane stability, and finally the conclusions of this initial stage of the second phase of this research study.

Chapter 2 reviews previous research studies. The Phase 1 experimental results are provided, along with relevant experimental studies on high height/thickness ratio RMSWs performed by other researchers. Out-of-plane instability evidence from past earthquakes and a review of international standard provisions addressing this failure mechanism are also presented.

Chapter 3 describes the design, construction, material properties, instrumentation and preparation of the test specimens. The reversed-cyclic loading test procedure and the associated loading
protocols are presented in this chapter, while additional details on the specimen design, construction, and experimental test setup are provided in Appendices A and B.

Chapter 4 presents and discusses the behaviour and results of reversed-cyclic testing of the two wall specimens which were subjected to a displacement-controlled reversed cyclic loading, with incrementally increasing displacements until the failure. Chapter 4 compares the test results for the two specimens and examines the effect of axial load and its effect on the response of the test specimens. The behaviour of each specimen is examined to identify possible additional factors which may impact the out-of-plane stability of RMSWs.

Chapter 5 evaluates the current state of an analytical model simulating the important compression zone of a RMSW subjected to seismic loading. The model comprises of a vertical section of a RMSW which is loaded in uniaxial tension and compression similar to the tests performed in Phase 1 of this research program. Chapter 5 also summarizes the key research findings, describes how these findings may influence future design practices, and provides recommendations for future research.
CHAPTER 2

Literature Review

Several experimental research studies on the behaviour of RMSWs subjected to reversed cyclic, in-plane loads have been performed by other researchers. Most studies have explored the overall flexural and/or shear response of RMSWs subjected to in-plane lateral loading, while the current study explicitly examines the flexural response with a focus on the out-of-plane instability. Nonetheless, the information presented by these studies was found to be relevant for planning of the current experimental study.

An extensive review of previous experimental research on RMSWs was conducted by Azimikor et al. (2011) which proved useful during the compilation of this review as did another compilation of prior research performed by the Federal Emergency Management Agency (FEMA 306) in 1999. The former review concentrated on research studying the seismic performance of RMSWs exhibiting a flexural response whereas the latter review was of a broader and all-encompassing nature.

The specific objectives of the literature review presented in this chapter are described below as:

- To obtain a better understanding as to the existing state of knowledge of seismic response of RMSWs based on previous experimental evidence
- To compile a set of key parameters influencing the out-of-plane instability of RMSWs subjected to in-plane loads
To use this set of parameters to design the test specimens for the current experimental study
- To identify knowledge gaps where additional research is required

All technical information is presented in a condensed format in order to summarize the most relevant results while maintaining brevity. The experimental results of previous research studies present a backdrop for the results of the current study. Using the data from these prior studies in combination with that of the current study aids in the identification of the key parameters influencing the out-of-plane instability failure mechanism in RMSWs.

Literature review presented in this chapter begins with a discussion on experimental testing performed in Phase 1 of this research program, including a thorough description of the behaviour of the test specimens experiencing out-of-plane instability and the test results (Azimikor, 2012; Azimikor et al. 2012). Further on, two main groups of experimental research studies (tests) on RMSWs were discussed. The first group of tests focuses on the overall response of high h/t ratio RMSWs subjected to in-plane lateral loading. The tests investigated in this group have been selected based on their h/t ratio and the method of loading. Most wall specimens from this group of tests experienced either a flexural or a combined flexural and shear failure mechanism. The second, much smaller, group includes tests where specimens exhibited out-of-plane instability. The number of tests in this group is relatively limited and the objective of the current study was to expand the research in this area. The chapter concludes discussing evidence of out-of-plane instability from recent earthquakes and a brief review of provisions of Canadian masonry and concrete design standards related to out-of-plane instability.

2.2 Experimental Studies on Uniaxial Reinforced Concrete and Reinforced Masonry Specimens Subjected to Reversed Cyclic Tension and Compression

This section reviews some previous experimental research conducted on reinforced concrete and reinforced masonry specimens subjected to reversed cyclic tension and compression. Several experimental studies on reinforced concrete are first briefly mentioned followed by a more
detailed summary of the reinforced masonry specimens tested in the first phase of this research program.

Reinforced concrete shear walls and RMSWs are generally characterized by similar seismic responses and thus noting research conducted on similar reinforced concrete specimens can be useful. This is especially the case when examining the out-of-plane instability failure mechanism which is shared between the two types of walls. This failure mechanism has been observed in any many tests on reinforced concrete shear walls (Valleñas, Bertero and Popov, 1979; Goodsr, 1985; Thomsen and Wallace, 2004) as well as the RMSWs discussed in the next sections of this chapter.

Phase 1 of this study was the first experimental study on uniaxial reinforced masonry specimens despite that this type of research on reinforced concrete dates back to the mid-1980s. After examining the out-of-plane instability behaviour in reinforced concrete shear walls, Goodsir (1985) conducted a study on a series of nine column specimens built with slenderness ratios ranging from 5.0 to 7.0. Chai and Elayer (1999) also conducted an experimental study to assess the lateral stability of reinforced concrete shear wall end zones by testing axially loaded reinforced concrete columns. Cycles of axial tension and compression were applied to the column specimens to derive an analytical model to be discussed later in Section 2.6. Drawing the conclusions of Chai and Elayer (1999), Acevedo and Moehle (2010) performed similar tests on non-special boundary elements of shear walls following the February 2010 Chile earthquake. This study concluded that when reinforced concrete boundary elements or columns are subjected to axial load reversals, out-of-plane instability becomes a primary failure mechanism.

The first phase of this research program, performed by Azimikor (2012), was focused on understanding the out-of-plane instability phenomenon in RMSWs by simulating response of wall end zones by uniaxial (column-like) specimens. Figure 2-1 displays the experimental test setup of Phase 1 and Figure 2-2 shows one of the column specimens during testing. In total, five full-scale reinforced masonry specimens with an h/t ratio of 27 and varying reinforcement ratios were used for this study. The specimens were subjected to reversed cyclic compression and tension loading. The test results proved to be valuable in understanding the out-of-plane instability failure mechanism and identify key factors which influence its development; the
results were useful for the design of Phase 2 wall specimens. However, these specimens were not able to simulate the actual boundary conditions along the height of the wall end zone and did not take into account the effect of the strain gradient along the wall length.

Figure 2-1   Phase 1 experimental test setup (Azimikor 2012)
All test specimens exhibited out-of-plane displacements leading to lateral instability. It was observed during the testing that the instability mechanism heavily revolves around the development of large tensile yield strains in the vertical reinforcement prior to compression loading. As the specimens experienced high tensile strains, uniformly distributed horizontal cracks began to form over the height of the plastic hinge region. Upon reaching significant
tensile strains beyond yielding at the point of maximum tension for each cycle, widths of uniform horizontal cracks developed along mortar bed joints would reach their maximum for the cycle. When the applied tension load was removed, the crack width would significantly reduce, but the cracks would not completely close due to the presence of inelastic tensile strains in the reinforcement. The specimens were then loaded in compression and the reinforcement was forced to sustain the entire compressive load across the cracks and transfer it to the grouted masonry core. At that stage, the vertical reinforcing bars behaved as though they were laterally unsupported over the entire height of the plastic hinge region, and their low lateral stiffness and insignificant moment of inertia caused out-of-plane displacements in the specimen. This was effectively the case as the reinforcing bars were unsupported over numerous discrete crack locations along the plastic hinge region. If a number of bars were experiencing this exposure at crack locations (as was the case in all of the Phase 1 column specimens), the reinforced masonry would become vulnerable to instability. These out-of-plane displacements in turn led to transverse rotational cracks occurring at the bed joints. If the initial horizontal flexural cracks developed during the tension loading part of the cycling were not sufficiently wide, the out-of-plane lateral displacements would decrease, because the combination of the lateral displacements and corresponding rotations of the horizontal flexural cracks would help achieve contact between the masonry face shells on either side of the crack; this would lead to the development of masonry compression forces across the cracked section. If out-of-plane displacement were less than the critical out-of-plane displacement for the specimen, these vertical masonry compressive forces would produce a stabilizing bending moment across the cracked section, thereby preventing the occurrence of out-of-plane instability in the specimen and restoring its original vertical alignment. However, if the flexural cracks were sufficiently large, the out-of-plane displacements would exceed the critical limit and precipitate failure due to out-of-plane instability.

The following relevant findings were made related to the out-of-plane instability:

1. Out-of-plane instability occurred when significant tensile strains (on the order of 3.2 to 6.3\(\Delta_y\)) developed in the reinforcing bars across uniform horizontal cracks (where \(\Delta_y\) denotes yield strain in steel reinforcement).
2. Uniform horizontal cracks need to develop along mortar bed joints before out-of-plane instability could occur. These cracks also need to be wide enough to cause the bars to lose lateral support over the height of the horizontally cracked portion of the specimen.

3. Once plastic tension strains developed in the reinforcement, compression stresses in the reinforcing bars would lead to out-of-plane displacements of the entire specimen, eventually triggering lateral instability once a critical tension strain was induced in a previous cycle.

4. The critical out-of-plane displacement that needed to be reached before the out-of-plane instability takes place was found to be typically equal to approximately half the specimen thickness.

5. The reinforcing bar diameter needed to be large enough to prevent local buckling before global instability could occur.

### 2.3 Experimental Studies on High Height/Thickness Ratio RMSWs Subjected to In-Plane Lateral Loading

This section presents an overview of relevant experimental studies on high h/t ratio RMSWs subject to in-plane lateral loading. All test specimens in this section are characterized by h/t ratios greater than 14 (the most stringent h/t limit imposed by the CSA S304.1-04 provisions). This review presents a background and characterizes the general behaviour and the types of failure mechanisms for RMSW specimens which typically showed flexural or a combined flexure and shear response. Studies by Elshafaie et al. (1999), Shedid et al. (2005), Miller et al. (2005), and Nolph (2012) include specimens which did not experience out-of-plane instability, however these results can be used to identify criteria for preventing out-of-plane instability of RMSWs characterized by ductile flexural response.

There have been only three experimental studies on high height/thickness ratio RMSWs subjected to in-plane loading which reported observations of out-of-plane instability. Two of these studies are relatively recent (Shedid et al. 2008; 2009), and the third study was performed in 1992 by He and Priestley. It should be noted that these studies did not have the objective of
observing out-of-plane instability, however a few of the test specimens did experience out-of-plane buckling in the wall web. The data and observations from these studies were relevant for developing experimental program for this study. It should be noted that all specimens discussed in this section have smaller h/t ratios than the wall specimens examined in the current study.

The table on the following page (Table 2-1) summarizes details of relevant past experimental testing programs on RMSWs. The summary focuses on the specimen responses as well as their design parameters important to the current research. The tabulations are placed in the order that they appear in the remainder of this section. From left to right, the reference program is displayed followed by the individual specimen names, specimen height, unsupported specimen height, specimen thickness, aspect ratio (height/length), degree of grouting, flexural reinforcement ratio, masonry compressive strength, degree of axial loading, degree of shear loading, yield drift ratio, ultimate drift ratio and type of failure (F - flexural, F/S - flexural/shear).

The aspect ratio (height/length) was calculated using the height of the load application relative to the base of the wall specimen, \( h \), and dividing this by the total wall length, \( l_w \). The h/t (height/thickness) ratio however was determined using the laterally unsupported height of the wall specimens, \( h_u \). Therefore in the cases of multistorey wall specimens, only the first storey height was used in the calculation of the parameter. The degree of grouting is displayed as the (net area, \( A_n \)/gross area, \( A_g \)) of the masonry where net area subtracts the ungrouted area of the masonry cells. The flexural reinforcement ratio, \( \rho_f \), was calculated as the vertical reinforcement placed in the wall end zones. Net area was used in the determination of degree of axial loading. The degree of shear loading utilized the shear area \( A_v = 0.8A_g \). The maximum lateral load capacity of the wall specimen was used as the lateral wall strength \( V_{max} \).

To be consistent in the reporting of the yield drift \( \Delta_y/h \) and the \( \Delta_u/h \), were determined according to a procedure. The yield displacement, \( \Delta_y \), was obtained by extending a horizontal line from \( V_{max} \) to the vertical axis and laying out a line from the origin through the point on the hysteretic curve corresponding to lateral load capacity of 0.7\( V_{max} \). The intersection of these two lines was by definition the yield displacement as illustrated in Figure 2-3. The point on the post-peak hysteretic curve corresponding to a lateral load capacity of 0.8\( V_{max} \) was used to obtain that ultimate displacement, \( \Delta_u \).
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<td>$\rho_f$ (%)</td>
<td>$f'_m$ (MPa)</td>
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The following summaries are presented for additional information for each of the research studies presented in Table 2-1.

Elshafie et al. (1999) tested eight wall specimens composed of one-third-scale replica of 150 mm hollow concrete block masonry units. The tests were done using monotonic lateral loading at the top of each wall specimen with zero axial load application. The walls specimens had a maximum h/t ratio of 26.7 and a minimum h/t ratio of 19.3. Of the set of eight wall specimens, only one specimen was built with the higher h/t ratio. This specimen was also built with an aspect ratio of 1.0 whereas the other seven specimens were built with aspect ratios of 0.7. The specimen with the higher h/t ratio as well as four of the other specimens were reinforced with a 0.34% vertical reinforcement ratio and the remaining two specimens were had vertical reinforcement ratios of roughly double that (0.67%). All walls responded primarily in a flexural manner with the formation of plastic hinge lengths at the member ends. There were no observations of out-of-plane instability reported. (Elshafie et al. 1999)

Shedid et al. (2005) tested three full-scale, 200 mm thick, fully-grouted, standard concrete block masonry wall specimens. The tests involved the application of lateral reversed cyclic loading at the tops of the wall specimens with an absence of vertically applied axial stress. All three walls had an h/t ratio of 19 and an aspect ratio (height/length) of 2.0 to precipitate a flexural response. Two of the wall specimens were reinforced vertically with a vertical reinforcement ratio of 1.31% and horizontally with a horizontal reinforcement ratio of 0.26%. The third wall specimen had just under half of the reinforcement with a vertical reinforcement ratio of 0.73% and a horizontal reinforcement ratio of 0.13%. The response of all wall specimens was dominated by flexure with the formation of flexural horizontal cracking in the plastic hinge length and toe crushing at the wall toes. Some buckling of the reinforcement in the end zones of the wall specimen was observed after significant face-shell spalling had occurred. After the spalling, a void in the grouting during construction was exposed and it was determined that the reinforcement buckling was influenced by this fact. The empty cells extended up nine courses from the specimen footing creating a long unsupported length for the vertical reinforcement located in these cells. The buckling however occurred only after the occurrence of significant toe crushing and face-shell spalling. (Shedid et al. 2005)
Miller et al. (2005) tested four full-scale, 200 mm thick, fully-grouted, standard concrete block masonry wall specimens. All wall specimens were subjected to in-plane lateral loading at the top of their height and two of the specimens were also loaded axially with a compressive stress of 1.0 MPa. Two of the wall specimens had an h/t ratio of 15.8 and the other two specimens had an h/t ratio of 10.5. All four of the wall specimens were squat walls with an aspect ratio (height/length) of 1.0. The vertical wall reinforcement was consistent for all of the wall specimens with vertical reinforcement ratio value of 0.79%. The horizontal reinforcement ranged from a horizontal ratio of 0.13% to 0.07%. The response of the wall specimens was characterized first through significant diagonal shear cracking followed by flexural cracking along the bed joints and crushing at the toes of the specimens. There were no observations of reinforcement buckling or out-of-plane instability reported during the testing despite an accidental out-of-plane load applied to one of the specimens during the tests. (Miller et. al 2005)

Nolph (2010) performed tests on six full-scale, 200 mm thick, standard concrete block masonry wall specimens. One wall specimen was fully-grouted and used a comparison for the remaining five only partially grouted wall specimens. The grout in these latter specimens was contained in only the cells which contained the vertical reinforcement. This provided greater stability to the wall reinforcement when it was subjected to compression. The specimens were loaded laterally in-plane according to a reversed cyclic loading protocol. The loading continued until the specimens recorded a drop of 20% of their peak lateral load capacity. The h/t ratios for all wall specimens was 14.7 and the aspect ratios (height/length) were identical at 0.92. The specimens were reinforced with a vertical reinforcement ratio of 0.45% of the total area. The horizontal shear reinforcement ranged between 0.085% to 0.169%. Due to the combination of the low vertical reinforcement ratio and relatively high horizontal reinforcement ratio, the wall specimens from this testing exhibited primarily shear failure mechanisms through diagonal step cracking in the central portions of the walls with a minor amount of vertical cracking at the wall toes. (Nolph 2012)

Minaie et al. (2010) conducted tests on four full-scale, 200 mm thick, standard concrete block masonry wall specimens. The wall specimens were partially grouted and specially reinforced to determine the appropriateness of current seismic design provisions for partially-grouted RMSWs. The provisions of the Masonry Standards Joint Committee (MSJC) code were used for the
specimen construction. The wall specimens were subjected to in-plane reversed cyclic displacements at the top of their height. Two of the specimens were also subjected to an axial load creating a compressive stress of 0.7 MPa with cantilever boundary conditions while the other two specimens had fixed-fixed boundary conditions and were not loaded axially. A number of test variables were modified in the test matrix of the four walls including the mortar formulation, level of axial stress, and the boundary conditions. All of the wall specimens had an h/t ratio of 13.9. Two of the wall specimens had an aspect ratio of 1.37 and the remaining two had an aspect ratio of 0.6. All wall specimens were vertically reinforced at a reinforcement ratio of 0.15%. The two specimens which were loaded axially displayed a flexural, toe-crushing failure mechanism. The remaining two specimens failed due to a sliding shear failure mechanism. (Minaie et al. 2010)

He and Priestley (1992) conducted tests on four full-scale T-shaped wall specimens to investigate how the seismic response of RMSW was influenced by the vertical reinforcement ratio, flange width, and concrete confinement. The tests consisted of loading the wall specimens both laterally in-plane at their top in a reversed cyclic manner, as well as axially in compression. The axial loading was held constant at 0.06f'm throughout the tests. All of the wall specimens had identical and high h/t and aspect ratios. The h/t ratio was 25.6 for all specimens and the aspect ratios were 3.1. The wall reinforcement ratios were either 0.23% or over double that at 0.50%. The horizontal reinforcement was the same for all of the wall specimens and had a reinforcement ratio of 0.44%. The response of the wall specimens consistently behaved in flexure. As a result, flexural mechanisms governed their failure behaviours. For three of the wall specimens, the yielding of the reinforcement when the web was in tension, precipitated a tendency for the vertical reinforcement to buckle. The fourth wall failed due to a flexural toe-crushing mechanism. One of the other walls, after buckling of the vertical reinforcement had occurred, experienced flexural out-of-plane wall buckling at the end of its web. The web confinement reinforcement had been subjected to high levels of residual inelastic tensile strains at this point the testing. This wall specimen had the highest vertical reinforcement ratio and lowest axial load of the tests and was the only specimen to exhibit out-of-plane instability. (He and Priestley 1992)
Shedid et al. (2008) tested six full-scale, 190 mm thick, fully-grouted, standard concrete block masonry wall specimens. Under investigation was the possibility of achieving a high level of ductility by flexural yielding in the wall specimens. The focus was on the effects of amount and distribution of vertical reinforcement and the level of axial compressive stress on the inelastic behaviour and ductility of RMSWs. All of the wall specimens were loaded laterally in-plane at their top. Four of the six specimens were not loaded axially and the remaining two were loaded at 260 kN and double that at 520 kN respectively. The wall specimens had identical h/t and aspect ratios of 18.9 and 2.0 respectively. The vertical wall reinforcement ranged between reinforcement ratios of 0.29% and 1.31%. The horizontal reinforcement ratios ranged between 0.08% and 0.26%. The response of all specimens exhibited flexural behaviour characterized by horizontal cracking along the bed joints in the lower courses, face-shell spalling, and bar buckling and fracture at the wall end zones in the final cycles. One specimen of interest experienced a degree of out-of-plane buckling of the compression zone of the wall in the first five courses from the wall base at the conclusion of the testing. There was no compression loading applied to this specimen and it was built with the highest reinforcement ratio for both shear and flexure. (Shedid et al. 2008)

Shedid et al. (2009) performed additional testing on seven half-scale replica of 190 mm thick, fully-grouted, standard concrete block masonry wall specimens. The study was evaluating the flexural response of rectangular RMSWs and comparing it to that of walls with boundary elements and flanges. The wall specimens modelled two- and three-storey high RMSWs, incorporating slabs at the top of each "storey" level. Two control specimens were simply rectangular walls but the remaining five incorporated different types of boundary elements. The boundary elements were located at the wall end zones and varied between a single-block flange, a two-block flange, and a pilaster unit flange, each with a different vertical reinforcement ratio and layout. Lateral in-plane loads were applied at the top of each wall specimen. Axial loads were constant for each test at 160 kN and were applied at the top of the wall specimens by high-strength prestressing rods anchored to the base. The wall specimens maintained a constant length but had a variable height. The corresponding h/t and aspect ratios varied between 14 and 21, and 1.5 and 2.2 respectively. The vertical reinforcement ratios ranged from 0.55% to 1.17%. The horizontal reinforcement ratios ranged between 0.3% and 0.6%. The response of the wall specimens was predominantly flexural as was the final failure mechanism. The failure
mechanisms were consistently characterized by extensive yielding of the vertical reinforcement along horizontal flexural cracks followed by toe crushing and splitting at the wall toes. The wall specimens experienced extensive widening of the vertical cracks in the tension end zones and crushing at the wall toes along with buckling of the outermost vertical reinforcing bars at the compression zone end. Much of this damage was confined to the flanges of each specimen. The two rectangular wall specimens built without boundary elements experienced out-of-plane displacements accompanying the bar buckling in their compression toe regions. These out-of-plane displacements only lead to an out-of-plane instability failure mechanism in one wall specimen. This specimen was the most highly reinforced against both shear and flexure. (Shedid et al. 2009)

Based on the review of experimental studies on high height-to-thickness ratio RMSWs, the following conclusions can be made:

1. **Specimens which did not experience out-of-plane instability**
   
The results of this review indicate that high h/t ratio RMSWs (characterized by h/t ratio equal to or greater than 14), when subjected to in-plane lateral loading, exhibit flexural behaviour. When properly reinforced to produce a ductile response, these walls fail primarily in flexure rather than shear. As the h/t ratio decreases, the failure mechanisms generally shift toward shear. Flexural failure mechanisms are characterized by the yielding of vertical reinforcement toe crushing. Shear mechanisms are characterized by diagonal tension cracking and sliding along the bed joints. Often is the case where the failure mode of RMSWs is a combination of flexural and shear mechanisms. There is no evidence of global out-of-plane instability in these studies, however buckling of the vertical reinforcing bars occurred as a result of insufficient lateral support for the wall specimen(Shedid et al. 2005).

2. **Specimens which did not experience out-of-plane instability**
   
   Out of three experimental studies discussed in this section, only one specimen per study exhibited out-of-plane displacements and ultimately the failure occurred when specimens experienced out-of-plane instability. The specimens which displayed this behaviour had the highest vertical and horizontal reinforcement ratios within their respective tests.
matrices; a similar conclusion was made by Azimikor et al. (2011). It would also appear that relatively high axial stress level was possibly a requirement for this behaviour. This interesting as the findings of the current study suggest otherwise.

2.4 Out-of-Plane Instability Evidence from Past Earthquakes

Occurrence of the out-of-plane instability failure mechanism in RMSWs subjected to in-plane loading has not been reported in the aftermath of past earthquakes. However, there is evidence of this failure mechanism in reinforced concrete shear walls.

In the aftermath of the February 27, 2010 Maule, Chile earthquake out-of-plane instability was observed in the first storey of multi-storey reinforced concrete shear wall structures. Figure 2-1 shows an example of this failure mechanism, which occurred primarily in the web zones of T- and L-shaped wall sections. According to Moehle et al. (EERI, 2010), large cyclic strain demands imposed on the vertical reinforcement due to seismic loading, combined with relatively large axial stress level, might have produced lateral instability. In those instances, lateral instability occurred prior to concrete cover spalling and local bar buckling. Thickness of the walls succumbing to this failure mechanism were found to be generally small (on the order of 150 to 200 mm), especially considering the significant building height and large h/t ratio for unsupported wall height.

Azimikor et al. (2012) performed a review of seismic design provisions of international masonry and concrete design standards prior to the Phase 1 experimental investigation. This review showed that international standards often omitted h/t ratio restrictions for RMSWs, and when they were existent, they were considerably less stringent than the Canadian Masonry Design Standard (CSA S304.1-04). Although this out-of-plane instability had not been adequately understood at the time of their ratification, despite past research dating back to the 1970s, provisions in preventing it have already been included in the Canadian design standard for reinforced concrete structures (CSA A23.3-04) among other standards and codes.

For example, U.S. Building Code Requirements and the Specification for Masonry Structures (TMS 402-11/ACI 530-11/A SCE 5-11) does not contain restrictions related to the h/t ratio, however there are requirements for wall boundary elements for walls subjected to higher axial loads and correspondingly larger compression zones (Cl.3.3.6.5.3a). If however, the axial loads are low (< 0.1A_{nf_m}) this requirement is waived.

The New Zealand Masonry Design Standard (NZS 4230:2004) includes restrictions on the h/t ratios for ductile RMSWs (ductility ratio from 2.0 to 4.0). Ductile RMSWs less than three storeys high may have a maximum h/t ratio of 20 (Cl.7.3.3), while taller RMSWs are limited to a maximum h/t ratio of 13.3. It should be noted that there is an escape clause which may be used by a ductility check on the c/l_w ratio, where c is the length of the compression zone.

It is noteworthy that both the Canadian concrete design standard (CSA A23.3-04) and the New Zealand concrete design standard (NZS3101:2006) include provisions for preventing out-of-plane instability in reinforced concrete shear walls. In the case of CSA A23.3-04, h/t restrictions date back to 1984. These provisions place restrictions on the minimum thickness of reinforced concrete shear walls but these restrictions are often related to other parameters such as size of the compression zone. However, guidelines for seismic rehabilitation of existing buildings in NZ and
USA, that is, NZSEE, 2006, and ASCE 41-06 (ASCE, 2006) do not contain provisions regarding the out-of-plane stability of reinforced concrete shear walls. It should be noted however, that recent New Zealand guidelines (Practice Note No. 30.3: Nonlinear Analysis Acceptance Criteria for Existing Reinforced Concrete Buildings) do address out-of-plane stability and have provided provisions involving the limiting plastic rotation associated with out-of-plane buckling.

2.6 Analytical Studies on Reinforced Masonry and Reinforced Concrete Shear Walls End Zones

Two primary analytical studies based on both reinforced concrete columns and reinforced masonry columns were identified to aid in the development of a future analytical model on a RMSW end zone and eventually, a RMSW model. Chai and Elayer (1999) performed an important analytical study related to the out-of-plane stability of reinforced concrete shear wall end zones. This study was influential in the development of the analytical model produced by Azimikor et al. (2012), the other very important analytical study on shear wall end zones. The model was shown to perform very well with the column specimens tested in that research.

As mentioned in Section 2.2, Chai and Elayer (1999) conducted an experimental study on the reversed cyclic axial loading of reinforced concrete columns. The experiments showed that the cracking of the column specimens followed a uniform spacing corresponding to the same spacing as the transverse reinforcement. The uniformity in the crack spacing made this study of particular interest for the similar crack location uniformity found in reinforced masonry where the crack locations are primarily found along the bed joints. Based on the results of their tests and earlier work by Paulay and Priestley (1993), an analytical model focused on estimating the maximum tensile strain after reversed cyclic loadings which would precipitate lateral instability was formulated.

Following the experimental testing of Phase 1 of this research program, Azimikor et al. (2012) developed an analytical model estimating axial strains which would initiate an out-of-plane instability failure mechanism in reinforced masonry columns subjected to axial reversed-cyclic loading. Despite neglecting the effect of the strain gradient factor occurring in shear walls as opposed to columns, the model was shown to effectively characterize the lateral instability mechanism of axially loaded masonry columns. The results of this model could then possibly be
used to represent an idealized end zone of an RMSW which is a primary objective of the future phases of this research program.

Azimikor et al. (2012) observed that all of the column specimens tested behaved in a similar manner with regards to out-of-plane stability. The experimental results indicated that the out-of-plane instability was a predictable mechanism. Drawing upon the findings of previous research by Paulay and Priestey (1993) and Chai and Elayer (1999), Azimikor et al (2012) developed a relationship for the general behaviour of reinforced masonry columns subjected to cyclic axial tension and compression loading with regards to out-of-plane instability. This relationship may be summarized in Figure 2-4. Important to note in the diagram, point $f$ only occurs in the loading cycle corresponding to out-of-plane instability.

![Figure 2-4 Axial reversed cyclic response of reinforced masonry columns (Azimikor 2012)](image-url)
From Figure 2-4, the path from o to a' describes the behaviour of the column specimen as it is being loaded in tension, where point a indicates the point of tensile yielding of the vertical reinforcement following cracking of the mortar joints. After a' has been reached the specimen is loaded in compression until point h (or in the case of out-of-plane instability, point f). Point b indicates the point at which the elastic strain developed during the tension portion of the cycling has been recovered. During the transition from point b to c, the now-exposed vertical reinforcement must carry the entire vertical load applied to the specimen.

As the compression loading increases, the specimen follows one of two paths; either from e to d or c to f. Initially upon departing from point c, the specimen experiences out-of-plane displacements resembling buckling over the height of the plastic hinge. This is caused by the combined effects of uniform cracking and the insignificant lateral stiffness of the exposed vertical reinforcing bars.

In the case where the behaviour leads to point d, the transverse plastic rotation across the cracks increases to a stage where contact is achieved along one of the face shells at the extremes of the crack opening. The compression forces produced by this contact provide a restoring moment which acts to close the cracks further. This continues as the specimen advances to point h, passing through points e and g which represent the initiation of local face shell crushing and the onset of reinforcement compression yielding respectively. At point h, the cracks have now completely closed and the specimen is restored to its original plumbness in pure compression. Unloading may occur after this point.

If however, the cracks are sufficiently wide after point e, the specimen will continue along a path to point f. If the cracks remain open when a critical out-of-plane displacement is reached, the out-of-plane displacements will become unstable leading to instability of the specimen. It can be seen through a simple free-body-diagram that this critical out-of-plane displacement will usually be equal to half the specimen thickness (depending on the contact location of the boundary conditions). For the restabilization to occur by the restoring moment caused by face shell closure, this area of contact must not pass under a vertical line passing through the upper and lower specimen boundaries. If this is not the case there remains no tendency for the specimen to return to its initial plumbness.
The out-of-plane displacements cause rotations over the specimen height as shown in Figure 2-5 where $T$ and $C$ are the tensile and compressive loading respectively. The compression loading is divided into both the masonry, $C_m$, and steel, $C_s$, components ($C = C_m + C_s$). $\delta$ represents the out-of-plane displacements of a given course of the column. $\xi$ represents the ratio of $\delta$ to the total course thickness, $t$ ($\xi = \delta/t$). $\gamma$ is a fraction of the wall thickness corresponding to the masonry compression force lever arm length. For example, if only the extreme fiber of the masonry is in contact, the lever arm length is half the wall thickness (ie. $\gamma = 0.5$).

![Figure 2-5](image_url)

**Figure 2-5**  
In-plane elevation view of column specimen (Azimikor 2012)

Azimikor (2012) then went on to discuss two models used to characterize out-of-plane instability of reinforced masonry columns. The objective was primarily to determine what conditions
causes a masonry column to follow the path c-d-e or c-f. It appeared that there was significant importance on the size of the crack widths along the specimen's plastic hinge and whether they would close before the initiation of instability. This lead to the conclusion that the tensile strain in the vertical reinforcing bars was a critical parameter for out-of-plane instability. Therefore, the models focused on finding a way to calculate the minimum tensile strain needed during a half-cycle of tension which would lead to instability in the following half-cycle of compression.

In determining the maximum permissible tensile strain, $\varepsilon_{sm}$, while maintaining stability, three primary types of strain needed to be considered; the elastic recovery strain $\varepsilon_e$, the reloading strain, $\varepsilon_r$, and the residual axial strain at the onset of crack closure that remains throughout the plastic hinge height in the form of open but rotated cracks, $\varepsilon_p^*$. The strains are illustrated in Figure 2-6. The sum of these three strains is equivalent to the maximum tensile strain as shown in Equation 2-1.

$$\varepsilon_{sm} = \varepsilon_e + \varepsilon_r + \varepsilon_p^*$$

![Figure 2-6 Maximum permissible tensile strain in terms of its constituents (Azimikor 2012)]
The first two terms of Equation 2-1, $\varepsilon_e$ and $\varepsilon_r$, correspond to the strain related to the elastic recovery process and the additional strain required to cause critical out-of-plane displacements respectively. The elastic recovery strain was described as a portion of the elastic strain as shown in Equation 2-2 where $\eta_1 > 1.0$ (to take into account strain hardening). It is assumed that out-of-plane displacements continue while the cracks remain open due to the insignificant lateral stiffness of the vertical reinforcement. The crack closure, as a result of the sectional rotation, signifies the end of the reloading strain. Due to this definition, the reloading strain could be approximated using an idealized buckled shape.

$$\varepsilon_e = \eta_1 \varepsilon_y$$

With the assumption of fixed-fixed boundary conditions and a maximum amplitude of half the column thickness it was possible to determine the change in height following completion of the elastic recovery process. This height was then used to obtain a derivation of the elastic recovery as shown in Equation 2-3 where $\xi_m = \xi = 0.5$ for most cases. Important assumptions of homogeneous reinforcement laid in the center of the units and following a sinusoidal curvature distribution were noted.

$$\varepsilon_r = \frac{\Delta h}{h} = 1.25 \xi_m \left(\frac{t}{h}\right)^2$$

This final strain component, $\varepsilon_p^*$, there consists of the strain existing at the rotated bed joints after the beginning of crack closure. Two similar but different approaches were used to determine this strain component; first was to estimate residual strains based on an assumed bed joint rotation distribution and the second was to assume these strains based on an assumed curvature distribution with the maximum curvature occurring at midheight.

Noting that in a reinforced masonry element, residual strains existing at the onset of crack closure are concentrated mostly at the bedjoints, Azimikor was able to assume a bed joint rotation distribution based on an overall buckled shape and the boundary conditions. Figure 2-7 illustrates this assumed distribution. Relating the individual crack rotations and geometries to an overall downward displacement at the top of the specimen, it was possible to derive a total residual strain corresponding to the critical out-of-plane displacement.
The derivation of $\varepsilon_p^*$ was combined with the other strain terms to obtain a maximum permissible tensile strain. The result of the derivation is shown in Equation 2-4. The model was calibrated using the experimental testing of the reinforced masonry columns accompanying this phase of the research program. The overall results agreed quite well for all of the column specimens tested.

$$\varepsilon_p^* = 2 \xi_m t \left[ \left( \frac{1}{2H_b} \right) \alpha \theta_{\text{max}} + \frac{2\theta_{\text{max}}}{h} \left( 1 - \alpha \right) H_b \left( \frac{1}{4H_b} \right) \left( \left( \frac{h}{2H_b} \right) - 1 \right) \right]$$ 2-4
The other model involved assuming a curvature distribution profile over the height of the column specimen. This approach assumed that the total residual plastic strain that remains in the buckled shape of the specimen in the form of open cracks may be estimated based on the maximum transverse curvature, $\varphi_{max}$, at midheight of the plastic hinge. This maximum curvature was used to describe $\varepsilon_{p}^*$ as is shown in Equation 2-5. This requires another assumption that the bed joint cracks are equal in width over the height of the plastic hinge.

$$\varepsilon_{p}^* = 0.5t \varphi_{max}$$  \hspace{1cm} 2-5

Neglecting the actual relatively sinusoidal buckled shape observed during the experimental testing, Azimikor assumed a circular curvature distribution from a number of alternatives shown in Figure 2-8 due to its superior fit with the experimental data. The circular distribution of curvature led to the derivations of Equations 2-5 and 2-6. This model also performed very well with regards to experimental results.

$$\varepsilon_{p}^* = 4 \varepsilon_{sm} \left(\frac{t}{h}\right)^2$$  \hspace{1cm} 2-6

$$\varepsilon_{sm} = 5.25 \xi_m \left(\frac{t}{h}\right)^2 + \eta_1 \varepsilon_y$$  \hspace{1cm} 2-7

Azimikor proposed to use the models to estimate tensile strain demand limits which would limit the probability of out-of-plane instability. This required the use of ductility factors corresponding to specified ductility demands. A derivation of the tensile strain demand in the end zone of a RMSW in terms of a specified curvature ductility was completed and shown in Equation 2-7. This in turn was used to derive an equation to determine the minimum wall thickness required to prevent out-of-plane instability at a given ductility level shown in Equation 2-8.  

$$\varepsilon_{id} = 0.85 \left[ \frac{2h^2}{3l_p(2h - l_p)} (\mu_\Delta - 1) + 1 \right] (\varepsilon_{sy} + \varepsilon_{my})$$ \hspace{1cm} 2-8

$$t \geq \sqrt{\left[ 0.85 \left[ \frac{2h^2}{3l_p(2h - l_p)} (R_d - 1) + 1 \right] (\varepsilon_{sy} + \varepsilon_{my}) - \eta_1 \varepsilon_y \right] \frac{h^2}{2.625}}$$ \hspace{1cm} 2-9
2.7 Summary and Conclusions

The literature review presented in this chapter, including the results of Phase 1 of this research study and other prior experimental test results, as well as past earthquake evidence and a review of international standard and code provisions, provides a background for the current experimental study. Earthquake evidence and a review of international standards and code provisions demonstrate that out-of-plane instability may occur in reinforced concrete shear walls, thus provisions and criteria for preventing this mechanism have been included in concrete design standards. However, there has been a lack of substantiated evidence of this failure mechanism in RMSWs.

Design parameters are found to affect the occurrence of out-of-plane instability in RMSWs, including vertical and horizontal reinforcement ratios, the level of applied axial stress, the h/t ratio and the h/l aspect ratio. In order for the out-of-plane instability mechanism to take place, large inelastic tensile strains must be experienced in the vertical reinforcing bars at the wall end zones, and wide uniform horizontal cracks need to form over the specimen's plastic hinge height. This requires significant curvature ductility demands to be withstood by the masonry in order to
precipitate this failure mechanism. Work by Azimikor (2012) in Phase 1 helped to better quantify and understand the importance of these parameters.

Based on these conclusions, several design parameters were considered to be critical for triggering out-of-plane instability failure mechanism and were used to design the test specimens and loading protocols of the current experimental program.
CHAPTER 3

Experimental Program

To complete the objectives of this research, to identify the variables controlling the out of plane failure of RM walls during cyclic in-plane loading, an experimental program was considered necessary. Specimens and testing apparatus were specifically design to exhibit the out-of-plane instability failure mechanism described in the previous chapter. The absence of previous experimental studies performed with this same objective, other than those of the uniaxial column tests performed by Azimikor (2012), leaves a void of much needed test results required to make alterations to the stringent h/t ratio restrictions prescribed by the Canadian Masonry Design Standard (CSA S304.1-04). This chapter describes the experimental program in an overview. First to be presented here is the design, construction, and materials properties of the wall specimens followed by the design and construction of the experimental test setup, and finally the testing procedure and loading protocol. Details of the instrumentation and how it may be utilized to calibrate a future RMSW analytical model is introduced here as well.

3.2 Background

When a RMSW is subjected to seismic loading it experiences lateral in-plane loads arising from the inertial forces of the structure in addition to the permanent gravity loads applying axial compression on the wall from the floor stories above. Therefore, in a seismic event, axial loads, shears, and in-plane overturning bending moments will be transferred to the wall, which must in turn carry these loads to its base. As the overturning moment along the base of the wall alternates
direction, as is the case during a seismic event, the end zones of the wall experience correspondingly alternating cycles of compression and tension. This action is suspected to cause the end zones of the wall to be vulnerable to the out-of-plane failure instability mechanism of interest according to S304.1-04. Therefore, the primary objective of the experimental program was to subject wall specimens to this type of loading.

In addition, the wall specimens were designed to represent the bottom floor of a three to four storey building or the bottom portion of a tall-walled building. To make conclusions as to whether the S304.1-04 h/t ratio stipulations are excessively stringent, the wall specimens needed to possess h/t ratios equal to or greater than the h/t restrictions imposed by S304.1-04. Due to the nature of the out-of-plane instability failure mechanism, the wall specimens must be slender and behave so that a flexural failure mode governs over a shear failure mode. A final objective of the wall specimen design was to incorporate the findings of the first phase of this research project by Azimikor (2012) while also maintaining wall properties similar to that found in practice.

![Seismic loading on a bottom storey RMSW of a taller structure](image)

Figure 3-1   Seismic loading on a bottom storey RMSW of a taller structure
3.3 Specimen Description

In order to study this out-of-plane instability failure mechanism it was desired that the wall specimens to be tested were to fail due to a flexural mechanism rather than shear. Several measures were taken in an attempt to force this behaviour during the selection of the wall design parameters. The following sections describe the design, material properties, and construction of the two identical full-scale wall specimens studied in this phase of the study. A description of the design and consideration is presented first followed by the specific material properties of the elements used in the design and concludes with the construction details.

3.3.1 Specimen Design

Two identical full-scale wall specimens, W1 and W2, were the subject of this study and shown in Figure 3-2. The complete wall specimen dimensions and reinforcement layout are presented in Figure 3-3. The specimens were 2600 mm long and 3800 mm high (equivalent to 19 courses of masonry). They were constructed using standard concrete blocks (400 mm length x 200 mm depth x 140 mm thickness) laid in running bond, and mortar was used for face-shell bedding. The mortar mix used in the construction of the specimens consisted of 1: ½: 4½ proportions of Portland Type I cement, Type S hydrated lime, and fine aggregate by volume respectively to form a Type S mortar. Type S mortar is the most commonly used mortar for structural masonry with an above average typical 28-day strength of 12.4 MPa (1800 psi). The height and thickness of the wall were important parameters during the design of the wall specimens and were chosen to maximize the h/t ratio of the specimens. The height of 3800 mm was selected as it was the tallest specimen height (when combined with the 450 mm high footing) permissible in the UBC Structures Laboratory due to the restriction on the height of its 10 ton rail-crane. The thickness of 140 mm was selected because of the fact that it is the narrowest of the concrete block types used commonly in practice. Together these two parameters produced an h/t ratio of 27, almost double the limit for these types of walls prescribed by CSA S304.1-04. The specimen aspect ratio (height/length) ratio of 1.5 was chosen to ensure flexural behaviour characterized by the formation of plastic hinges at the base of the wall, a prerequisite for the out-of-plane instability mechanism. The vertical reinforcement consisted of one 15M reinforcing bar concentrated in the exterior two cells at each wall end, plus 10M bars spaced at 400 mm on centre along the wall.
length; this corresponds to a reinforcement ratio of 0.33% which is similar to that used in Canadian design practice. The vertical reinforcement distribution was determined considering the criterion to maximize the length of the compression zone, and the corresponding strains in the concentrated 15M reinforcing bars at the wall ends. The larger size of the exterior bars was a preventative measure based on the results of Phase 1 which suggest that the smaller 10M reinforcing bars may fail preemptively due to local bar buckling before out-of-plane instability can occur. The reinforcement was continuous up the wall height to eliminate the possibility of lap-splice failure at the base of the all during the testing. The capacity design approach was followed to avoid the possibility of shear failure in the specimen. The horizontal reinforcement was placed in bond beam blocks and consisted of 10M reinforcing bars at 200 mm spacing (each course); this corresponds to a reinforcement ratio of 0.36%. Alternating 180° hooks were provided at the ends of horizontal bars. The specimens were fully grouted with a commercially available regular coarse grout mix. The grout used in both pumping lift stages for each wall specimen had proportions of 1:3:2 by volume of Portland cement, fine and coarse aggregate respectively. The grout was transported to the UBC Structures Lab in a ready-mix truck and conformed to the requirements stipulated in the CSA Standard A179-04. Although CSA S304-01 requires only 12.5 MPa the Coarse Blockfill Grout grout had a nominal strength of 20 MPa. This is generally the case due to the higher strength mix being easier to pump. The top three courses were constructed using wider, 190 mm concrete blocks. These wider block were used to accommodate 700 mm long, 10M dowel reinforcing bars which were placed in the empty block cells to provide additional strength development near the ends of the continuous vertical bars. An additional horizontal 10M reinforcing bar was also provided in these top three courses to prevent local sliding shear failure along the interface between the loading beam and the wall specimen. This had been identified as a potential problematic issue based on previous similar experiments. An additional motivation for these wider blocks at the top of the wall specimens was to allow for the reuse of the loading beam during future testing on wall specimens incorporating the more typical and thicker standard 190 mm concrete blocks during the next phase of this study.
Figure 3-2  Test specimens W1 and W2 immediately after construction
The specimens were supported at their base by 810 mm x 450 mm x 2660 mm (width x height x length) heavily reinforced concrete footings, with all wall vertical reinforcement fully developed in the footing. The footings were designed to remain elastic during testing. Each of the footings were identically reinforced with 15M reinforcing bar transverse loops spaced at 50 mm and 8 longitudinal 15M reinforcing bars along their length. They were also fitted with 8 - 50 mm PVC pipes to permit the footing to be fixed through holes in the Structures Laboratory "strong floor." Additionally, 3 sets of two coupler holes were anchored within the footing by means of 25 mm anchors welded to 13 mm x 152 mm x 700 mm flat bar plates. These couplers permitted post-tensioning rods to be fastened from the loading beam to the footing during transportation of the specimens within the Structures Laboratory from the construction and testing locations. Figure 3-4 provides a cross-section diagram of the specimen footing.
Figure 3-3 Wall specimen - dimensions and reinforcement details
The specimen aspect ratio or height/length (h/l) ratio of 1.5 was chosen to ensure flexural behaviour characterized by the formation of plastic hinges at the base of the wall, a prerequisite for the out-of-plane instability. The wall h/t ratio of 27, significantly higher than the upper CSA S304 limit, was chosen to increase chances for out-of-plane instability during testing.

### 3.3.2 Specimen Construction

The test specimens were constructed at the same time within the Structures Laboratory facility at the University of British Columbia. The construction took place over the period of 2 weeks from the construction of the reinforced concrete footing to the final grouting of the finished masonry wall.

The reinforced concrete footings were the first components to be constructed. Two identical sets of formwork were built from ½" plywood, 2x4 lumber support frames, 4x4 support base, and lined with plastic to permit reusability. The tops of the formwork were built to 2 cm above the height of the footing. Three 2x4 lumber beams connected each side of the formwork to work
both as lateral restraints at the top of the formwork and also to prevent the coupler holes from filling with concrete during casting.

The reinforcement cage for the footing was assembled within the Structures Laboratory with the help of an experienced rebar layer. Each of the steel reinforcing loops were tied to the 8 longitudinal reinforcing bars using tie wire. Upon completion of the reinforcement cage, it was lowered into the footing formwork. The vertical reinforcement was then tied into the footing and supported by a supplementary frame built into the formwork. The support frame was constructed 2 m above the formwork base to stabilize the unsupported 4.2 m vertical reinforcing bars. The vertical reinforcement was attached to this frame by tie wire and at the same appropriate spacing as in the reinforcement cage. The vertical reinforcement was equipped with a single 180° hook on one end which was hooked around a transverse steel reinforcing loop at the appropriate spacing for the concrete block cells of the wall specimens. It was important to keep this reinforcement straight during the concrete pouring so that there were no stresses occurring within the specimens prior to testing. All reinforcement used within the wall and the footing came from the same heat with material testing results presented in the next section. Figure 3-5 shows the formwork and reinforcement cage placement prior to pouring the specimen footings.
Figure 3-5  Specimen footing formwork and steel reinforcement: (a) showing vertical reinforcing bars and supports, (b) closer view of footing reinforcement hardware, and c) close-up of formwork and reinforcement immediately prior to concrete pouring
The concrete was then poured directly from a ready-mix truck into the footing formwork in the Structures Laboratory. A large vibrator was used to sufficiently vibrate the concrete as to prevent voids from forming within the heavily reinforced and congested footing. Careful attention was paid however so as to avoid segregation of the aggregate. 10 concrete cylinders were cast during the construction of the footing. Curing measures were undertaken over the next 2 weeks prior to the construction of the masonry wall directly upon the footing. Each morning and evening the footing faces were sprayed with water and sealed with plastic. Figure 3-6 shows this stage of the construction.

![Figure 3-6 Specimen concrete footing immediately after concrete pouring and wrapping with plastic](image)

The masonry walls were constructed by professional masons atop of the now sufficiently hardened concrete footings. The concrete blocks were composed entirely of half, end, and full
bond beam units as shown in Figure 3-7. The three types of units were utilised on each course: full bond beam units for the interior of the walls and bond beam end and half units at the exterior of the walls. The full and end bond beam units had nominal dimensions of 140 mm x 200 mm x 400 mm and the half bond beam unit had nominal dimensions of 140 mm x 200 mm x 200 mm. All of the concrete block units were cast in the same batch. Many of the bond beam blocks were constructed using regular blocks by cutting two vertical slices into the webs of the block and knocking out the section between the slices.

Each concrete block unit was elevated to the top of the 4.2 m reinforcing bars and brought down such that the reinforcing bars were located within the cells of the block. The blocks were placed in a mortar bed approximately 10 mm thick and composed of Type S mortar. The mortar was mixed in a number of sets to maintain consistency in its workability. Mortar cubes were taken for every batch of mortar mixed and a mortar flow test was performed for each set of mortar cubes taken. Each course was placed in a running bond pattern for the specimens which is commonly used in concrete block masonry wall construction in Canada. One horizontal reinforcing bar with a single 180° hook on one end was laid within the notch of the bond beam blocks. The end of the wall length with the hook was alternated on every course. After the completion of each course, all excess mortar protruding from the joints was removed and the horizontal bed joints were finished with a curved steel jointing tool. This formed a uniform concave mortar joint finish.
typically observed in practice. The wall surface was finally brushed clean of all mortar spills and smears. Figure 3-8 demonstrates this overall procedure.

![Figure 3-8](image)

**Figure 3-8** Masonry wall construction process: a) layout of masonry units with vertical reinforcement extending from specimen footing, b) typical masonry block placement, and c) typical layout of horizontal reinforcement complete with 180° hook

This procedure was continued up until half the height of the wall specimen. At this point the first lift stage of the grouting began and all block cells were fully grouted with a commercially available regular coarse grout mix. The slender nature of the wall specimens created the possibility for instability if the grouting was done in a single lift stage because only the mortar bond strength could have been relied upon to support the full weight of the wall specimens. In addition to general stability, the hydrostatic pressure produced by grouting the full height all at once may well have exceeded the capacity of the relatively freshly mortared joints. Therefore the grouting was pumped in two stage lifts to ensure stability during the construction. The grout was transported to the Structures Laboratory by a ready-mix truck. Contractors mixed the grout prior to pumping as per standard practice. The grout was pumped directly into the wall cells using a grout pump truck. Slump tests were performed at the beginning and ending of grouting for each lift for both wall specimens. These results are presented in the following section. After the grout in the lower half of the wall had hardened the top half of the wall height was constructed in the
same manner. Upon completion of the concrete block placement at the full height the second lift stage of grouting commenced.

It is worthy of note that there was considerable care exerted to ensure the vertical wall reinforcement remained in the centre of each concrete block cell. The tests carried out in the previous phase of this experimental study did not take such care and this may have influenced the test results. Fortunately it was found that these efforts were fruitful and the reinforcement was located in the centre of their respective cell after the grout had hardened. The photograph of the vertical reinforcement at the top of the wall is shown in Figure 3-9.

![Figure 3-9](image)

**Figure 3-9** Reinforcement layout at top of wall specimen: a) reinforcement layout along the wall length, b) location of horizontal reinforcement $180^\circ$ hook, and c) location of vertical 10M dowel bars

During construction there were 10 two-block fully-grouted masonry prisms built for later material testing. Five were built for the first lift stage of grouting and five more for the second lift. In addition to the prisms, there were three grout blocks and 10 grout cylinders poured. The results of the material testing carried out on these specimens are described in the following sections.

The top three courses were constructed using the wider 190 mm x 200 mm x 400 mm bond beam blocks. Again there were three types of units utilised on each of these top three courses: full bond
beam units for the interior of the wall specimens and bond beam end and half units at the exterior of the specimens. The full and end bond beam units had nominal dimensions of 190 mm x 200 mm x 400 mm and the half bond beam unit had nominal dimensions of 190 mm x 200 mm x 200 mm. All of the concrete block units were cast in the same batch as the smaller width units. Many of these wider bond beam blocks were also constructed from regular blocks by cutting two vertical slices into the webs of the block and knocking out the section between the slices.

These top three courses were also placed in running bond and were centred in the middle of the 16 bottom courses with careful measurement for each block. This perfect centering was critical as there was only a small amount of face shell from the lower 140 mm blocks below which would overlap with the face shells of the 190 mm block above. This small overlap prevented leaking of the grout during pouring. Two horizontal reinforcing bars were placed in the bond beam notches in these courses, one on each side of the vertical reinforcement. These reinforcing bars had a single 180° on one end and were placed in opposite directions so as to provide a hook on each end of the wall length. A vertical 700 mm, 10M dowel reinforcing bar was placed in every cell which did not already contain one of the continuous vertical reinforcing bars from the footing. These dowel bars were formed with a single 180° hook on one end and this was used to rest the bar on top of the horizontal reinforcement. Figure 3-10 a) shows this transition at the top three courses of the wall specimens.

The masons took special care in maintaining the wall specimen plumbness as close to plumb as possible through means of a 5 ft level. Using a common industry practice, the level was placed on the wall face for each block placement to enable the masons to check if the wall was level and allowed for the position and orientation of each block to be adjusted accordingly. The final wall specimen plumbness was determined to be quite similar to that found in practice and more than sufficient so as not to affect the results of the tests.

After construction was complete, the wall specimens were each painted in a coat of white lime. The white lime was a mix purely from the combination of lime powder and water. An appropriate balance needed to be carefully found in order to achieve a desired thickness. Several iterations were required during the painting of the specimens. When the lime powder dries it appears substantially thicker than when it is in its liquid state. For this reason each of the specimens were required to have their surfaces scraped a number of times before they were
acceptable to write on. This was important for the crack marking to take place during the testing. Lime paint was utilized rather than regular latex paint due to its unique adhesive properties. The lime does not actually adhere to the wall and can be scraped off at will. The lime also has no tensile strength as opposed to normal paint. This enables cracks to readily form along the wall surface without interference from the lime while also aiding in the detection of cracks. As cracks form, their location is much more easily discerned on the white lime surface. Non-latex paints are generally preferred due to the difficulty in obtaining the appropriate thickness when using latex paints.

![Figure 3-10](image1.png)  ![Figure 3-10](image2.png)

**Figure 3-10** Photographs of the completed wall specimens: a) transition between the top three 190 mm thick courses and bottom sixteen 140 mm thick courses and b) wall specimen after lime painting
3.3.3 Specimen Material Properties

There were many material testing samples and specimens taken during the construction of the wall specimens. These were in the form of mortar cubes, grout cylinders, concrete cylinders, concrete block units, masonry prisms, and reinforcing bar samples. The concrete block units were taken from the same batch as the blocks used in the wall and the reinforcement came from the same heat as the reinforcement used in the wall specimens and footings. The materials testing was performed at the University of British Columbia's Structures Laboratory, the Structures Laboratory at the British Columbia Institute of Technology Department of Civil Engineering, and some tests at the block manufacturer's plant. Masonry material testing was performed following the procedures outlined in pertinent Canadian standards: CSA A165-04 for block testing, and CSA A179-04 for mortar and grout testing. Where the CSA standards were not relevant, the appropriate ASTM standards were utilized. The following sections summarize the key findings from the materials testing. Additional details may be found from the report by Levasseur and Yang (2012).

3.3.3.1 Masonry Prisms

The compressive strength of masonry is generally the most important mechanical property of masonry itself and was determined by the testing of fully grouted masonry prisms. Ten two-course high, stack bond pattern, masonry prisms were constructed by an experienced mason using the same mortar, grout, and concrete block units as used in the construction of the wall specimens. The mortar used in the face-shell bed-joint came from the last mix used for the wall specimens. The grout came from the second lift of pumping and was pumped directly into the masonry prism cells. The concrete blocks were from the same batch used in the wall specimens. To reflect the same construction environment as the wall specimens, the masonry prisms were constructed adjacent to the wall specimens in the UBC Structures Laboratory while construction of the wall specimens was nearing completion. All ten of the masonry prisms were fully grouted at the same time as the second lift of pumping for the wall specimens.

These prisms were used to determine the compressive strength, $f'_{m}$, of the grouted masonry used in the wall specimens. The masonry prisms were tested at the UBC Structures Laboratory on the Baldwin-Tate-Emery Universal Testing Machine which is composed of a control system, two
head shafts, and a loading table which was operated hydraulically and capable of applying 1780 kN in compression. Prior to testing, the prisms were cut to produce a flat surface using a concrete disk cutter to smooth the prism surface. Additional measures were also taken to ensure uniform loading along the prism surface by capping the top and bottom surfaces with "Hydrostone." Hydrostone is a high-strength, fast-setting gypsum cement commonly used for capping applications. Because the masonry prisms were slightly longer than the upper platen of the testing machine, a large 2" thick plate was placed on top of the masonry prism specimens to ensure that the entire top surface of the prism was under load. A piece of fiber board was set beneath and on top of the masonry prism in order to make certain uniform contact between the steel plate and the lower platen of test rig with the prism specimens. The compressive testing was performed in accordance with ASTM C140-99b Section 7. Figure 3-11 provides photographs of the masonry prism testing at the Basalite facilities and the UBC Structures Laboratory.

![Figure 3-11](image)

**Figure 3-11** Masonry prisms within test apparatus at: a) the facilities at Basalite and b) the UBC Structures of Laboratory

All masonry prisms failed exhibiting a similar brittle shear behaviour. Generally, the masonry prism's face shells split and spalled on either side of the grouted core after near vertical shear cracks separated the components. The loading was timed as described in ASTM C140 Section 6.4.3. The testing was conducted at masonry prism ages of 44 days, 118 days, 119 days, and 122 days with relatively negligible increase in strength for the latter tests. The average compressive strength of the masonry prisms was 21.2 MPa which is more than twice the requirement of the
CSA standard. Table 3-1 summarizes the masonry prism compressive test results, note that the slenderness correction factor was applied from an interpolation from CSA S304.1-04 Table D.1.

Table 3-1   Grouted concrete block prisms compressive strength summary

<table>
<thead>
<tr>
<th>Prism No.</th>
<th>Height (mm)</th>
<th>Area (mm$^2$)</th>
<th>Load (kip)</th>
<th>Load (kN)</th>
<th>Uncorrected $f_m$ (MPa)</th>
<th>Correction Factor</th>
<th>Compressive Strength (MPa)</th>
<th>Age (days)</th>
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<tbody>
<tr>
<td>1</td>
<td>388</td>
<td>55300</td>
<td>281</td>
<td>1259</td>
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<td>19.9</td>
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<td>55300</td>
<td>267</td>
<td>1197</td>
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<td>0.875</td>
<td>19.0</td>
<td>44</td>
</tr>
<tr>
<td>3</td>
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<td>55300</td>
<td>259</td>
<td>1159</td>
<td>21.0</td>
<td>0.875</td>
<td>18.3</td>
<td>44</td>
</tr>
<tr>
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<tr>
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<td>55300</td>
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<td>1295</td>
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<td>20.5</td>
<td>44</td>
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<td>6</td>
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<td>55554</td>
<td>307</td>
<td>1370</td>
<td>24.7</td>
<td>0.875</td>
<td>21.6</td>
<td>118</td>
</tr>
<tr>
<td>7</td>
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<td>55948</td>
<td>293</td>
<td>1307</td>
<td>23.4</td>
<td>0.875</td>
<td>20.5</td>
<td>118</td>
</tr>
<tr>
<td>8</td>
<td>388</td>
<td>55554</td>
<td>329</td>
<td>1466</td>
<td>26.4</td>
<td>0.875</td>
<td>23.1</td>
<td>119</td>
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<tr>
<td>9</td>
<td>384</td>
<td>55948</td>
<td>333.5</td>
<td>1487</td>
<td>26.6</td>
<td>0.875</td>
<td>23.3</td>
<td>119</td>
</tr>
<tr>
<td>10</td>
<td>389</td>
<td>55948</td>
<td>334</td>
<td>1488</td>
<td>26.6</td>
<td>0.875</td>
<td>23.3</td>
<td>122</td>
</tr>
</tbody>
</table>

Mean: 21.2 MPa
Standard Deviation: 1.8 MPa
Coefficient of Variation: 8.5%

3.3.3.2   Concrete Block Masonry Unit

The concrete block masonry units used for the construction of the wall specimens were manufactured at the facilities of Basalite in Vancouver, British Columbia, an alternate manufacturer than the manufacturer used in the for Phase 1 of this study. The manufacture of the blocks was done in accordance with the requirements of CSA A165 standard and all blocks came from the same batch to maintain uniformity in the wall specimens. Basalite provided the cross-sectional area of 44 in$^2$ for all 140 mm specimens and the correction factor for the use of the fiberboards was 1.1. The compressive testing was conducted at the Basalite plant using a Forney F401F-CPilot Universal Testing Machine and in accordance with CSA A165-04. The average compressive strength of 8 specimens was 49.8 MPa based on 8 block tests. The testing of the
concrete block masonry units is shown in Figure 3-12 and the results of the tests are summarized in Table 3-2, note that the area presented is the net area of a single full concrete block.

Table 3-2  Concrete block unit compressive strength summary

<table>
<thead>
<tr>
<th>Block No.</th>
<th>Area (in²)</th>
<th>Area (mm²)</th>
<th>Load (kip)</th>
<th>Load (kN)</th>
<th>Compressive Strength (MPa)</th>
<th>Correction Factor</th>
<th>Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>44.4</td>
<td>28645</td>
<td>417</td>
<td>1857</td>
<td>59.6</td>
<td>1.1</td>
<td>56.2</td>
</tr>
<tr>
<td>B2</td>
<td>44.4</td>
<td>28645</td>
<td>380</td>
<td>1693</td>
<td>53.3</td>
<td>1.1</td>
<td>48.4</td>
</tr>
<tr>
<td>B3</td>
<td>44.4</td>
<td>28645</td>
<td>375</td>
<td>1670</td>
<td>50.4</td>
<td>1.1</td>
<td>45.8</td>
</tr>
<tr>
<td>B4</td>
<td>44.4</td>
<td>28645</td>
<td>379</td>
<td>1689</td>
<td>50.3</td>
<td>1.1</td>
<td>45.7</td>
</tr>
<tr>
<td>B5</td>
<td>44.4</td>
<td>28645</td>
<td>353</td>
<td>1572</td>
<td>46.5</td>
<td>1.1</td>
<td>42.2</td>
</tr>
<tr>
<td>B6</td>
<td>44.4</td>
<td>28645</td>
<td>317</td>
<td>1411</td>
<td>43.4</td>
<td>1.1</td>
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<td>1522</td>
<td>51.6</td>
<td>1.1</td>
<td>46.9</td>
</tr>
<tr>
<td>B8</td>
<td>44.4</td>
<td>28645</td>
<td>263</td>
<td>1171</td>
<td>39.0</td>
<td>1.1</td>
<td>35.5</td>
</tr>
</tbody>
</table>

Mean: 45.0 MPa  
Standard Deviation: 5.4 MPa  
Coefficient of Variation: 12.0 %

Figure 3-12  Concrete block masonry unit testing at the facilities at Basalite
3.3.3.3 Grout

A secondary pump truck was used for pumping the grout to the top and mid-height of the wall specimens. From the pump truck a sample of grout was taken to perform a standard slump test. This was done to determine the workability of the grout used for each wall specimen and to ensure that voids would not occur in the lower sections of the wall height. The highly contested end cells of a RMSW, where the horizontal reinforcement is hooked, is often an area vulnerable to air pockets and voids. The first slump test was using grout from the first lift stage of each wall specimen and yielded a slump of approximately 282 mm. The second slump test was based on the grout from the second lift of each wall specimen and yielded a slump of 276 mm. This showed a general consistency in the workability from both lift stages.

Two types of test specimens were constructed in order to determine the average grout compressive strength of the grout used in the wall specimens. There were standard grout cylinders (200 mm height and 100 mm diameter) and grout prisms (75 x 75 x 150 mm high) cast during construction and in the same location of the wall specimens to best reflect the environment of the wall specimen grout.

Grout cylinders casting consisted of filling 200 mm high, 100 mm diameter standard plastic moulds during the wall specimen construction. The plastic moulds were greased beforehand to aid in specimen removal later on. Ten samples were taken overall. Five samples were from the first lift stage of grout and the remaining five were from the second lift stage of grout. Unlike the grout prisms, the grout cylinders represented artificially ideal conditions in which there is no moisture absorption due to the dry concrete block units.

The grout prisms did take into account the actual moisture absorption conditions found in the wall specimens as they were constructed using the pinwheel method. This method consists of placing four concrete block masonry units perpendicular to one another so that a 75 x 75 mm void is created in the centre of the "pinwheel" shape that the blocks form as shown in Figure 3-13. A 75 x 75 x 40 mm high wood block is placed at the bottom of this void. Grout is then poured into the remaining height of the void to form the grout prism. Some of the moisture from the grout located in this 75 x 75 x 150 mm high void is thus absorbed into the adjacent concrete block face shells in the same fashion as would be observed in the wall specimens.
The grout specimens were left in the UBC Structures Lab next to the wall specimens for three days before being removed from their moulds and transported to the moist curing room in the UBC Materials Laboratory. The specimens were set in a moist curing tank at a temperature of 20°C and 100% humidity. Figure 3-13 shows a picture of one of the compression cylinder tests.

![Grout cylinder compressive strength test setup at the BCIT Concrete Laboratory](image)

**Figure 3-13** Grout cylinder compressive strength test setup at the BCIT Concrete Laboratory

After 41 days the specimens were removed from the moist curing tank and their surfaces ground smooth using the UBC Materials Laboratory's concrete cylinder end grinder. This was done to ensure uniform distribution of the compressive load. Additionally, two rubber pads were placed at the top and bottom of the grout prisms during the testing. The compressive strength was determined in accordance with CSA Standard A179-04. The testing was conducted at the BCIT Concrete Laboratory using the standard Forney LT-704-3 Universal Testing Machine. The failure of the grout cylinders was characterized by vertical shear failure in all cases. The average grout compressive strength was 35.4 MPa (based on all 13 cylinders). The full results of the grout cylinder compressive strength test for both Pour 1 and Pour 2 respectively are summarized in Table 3-3 and Table 3-4.
### Table 3-3 Pour 1 grout cylinder compressive strength summary

<table>
<thead>
<tr>
<th>Cylinder No.</th>
<th>Height (mm)</th>
<th>Mass (kg)</th>
<th>Density (kN/m$^3$)</th>
<th>Load (kip)</th>
<th>Load (kN)</th>
<th>Compressive Strength (MPa)</th>
<th>Age (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1</td>
<td>194.0</td>
<td>3.5</td>
<td>22.45</td>
<td>15.20</td>
<td>68</td>
<td>8.6</td>
<td>31</td>
</tr>
<tr>
<td>1-2</td>
<td>196.0</td>
<td>3.6</td>
<td>22.86</td>
<td>44.73</td>
<td>199</td>
<td>25.3</td>
<td>31</td>
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<tr>
<td>1-3</td>
<td>197.5</td>
<td>3.6</td>
<td>22.95</td>
<td>52.36</td>
<td>233</td>
<td>29.7</td>
<td>31</td>
</tr>
<tr>
<td>1-4</td>
<td>198.5</td>
<td>3.6</td>
<td>22.89</td>
<td>49.65</td>
<td>221</td>
<td>28.1</td>
<td>31</td>
</tr>
<tr>
<td>1-5</td>
<td>199.0</td>
<td>3.6</td>
<td>22.90</td>
<td>48.33</td>
<td>215</td>
<td>27.4</td>
<td>31</td>
</tr>
<tr>
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<td>197.0</td>
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<td>22.93</td>
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<td>219</td>
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<td>31</td>
</tr>
</tbody>
</table>

Mean: 27.7 MPa  
Standard Deviation: 1.6 MPa  
Coefficient of Variation: 5.6 %

### Table 3-4 Pour 2 grout cylinder compressive strength summary

<table>
<thead>
<tr>
<th>Cylinder No.</th>
<th>Height (mm)</th>
<th>Mass (kg)</th>
<th>Density (kN/m$^3$)</th>
<th>Load (kip)</th>
<th>Load (kN)</th>
<th>Compressive Strength (MPa)</th>
<th>Age (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-1</td>
<td>197.5</td>
<td>3.6</td>
<td>22.69</td>
<td>74.52</td>
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<td>42.5</td>
<td>30</td>
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<tr>
<td>2-2</td>
<td>195.5</td>
<td>3.7</td>
<td>23.70</td>
<td>86.61</td>
<td>388</td>
<td>49.4</td>
<td>30</td>
</tr>
<tr>
<td>2-3</td>
<td>196.0</td>
<td>3.7</td>
<td>23.36</td>
<td>75.73</td>
<td>340</td>
<td>43.2</td>
<td>30</td>
</tr>
<tr>
<td>2-4</td>
<td>194.5</td>
<td>3.8</td>
<td>24.11</td>
<td>75.09</td>
<td>337</td>
<td>42.9</td>
<td>30</td>
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<tr>
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<td>197.5</td>
<td>3.8</td>
<td>24.21</td>
<td>76.13</td>
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<td>43.5</td>
<td>30</td>
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<tr>
<td>2-6</td>
<td>196.0</td>
<td>3.8</td>
<td>24.28</td>
<td>48.33</td>
<td>217</td>
<td>27.6</td>
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<td>76.70</td>
<td>344</td>
<td>43.8</td>
<td>30</td>
</tr>
</tbody>
</table>

Mean: 40.2 MPa  
Standard Deviation: 7.1 MPa  
Coefficient of Variation: 17.5 %

Unfortunately during the casting of the grouted prisms there was inconsistencies in the dimensions of the specimens. It was decided that the grout prism would not be tested due to their irregularity with standard testing procedures.
3.3.3.4 Mortar

The mortar was supplied by Target, another local Vancouver company. The mix was prepared by experienced masons and in accordance with CSA Standard A179-04. The water content of the mortar mix was monitored by the masons to ensure appropriate workability during the mixing.

There were many mortar batches used for each wall specimen due to the magnitude of mortar joints contained with each specimen. The batches were mixed in buckets by the masons and laid out on a plywood sheet during the application to process. Samples taken for the materials testing were taken just after mixing of each batch. Each batch was used relatively quickly in less than an hour and therefore the compressive strength of the mortar determined from the samples was determined to be representative of the mortar used within the wall specimens.

Mortar flow tests were used to determine the workability of the mortar used in the wall specimens. These were measured on-site immediately following the mixing of the mortar and in accordance with CSA Standard A179-04. The Type S mortar used in the wall specimens needed to be highly workable. The spread of the mortar on the test apparatus was measured to determine the diameter of the mortar flow required for the reading. Three mortar flow tests from different batches of mortar yielded an average mortar flow of 178 mm. This corresponds to a diameter increase of 78%, which is higher than specified in CSA Standard A179-04 (0-15% increase). This indicates a high workability deemed satisfactory by the masons. Figure 3-14 demonstrates a mortar flow test in progress.
Mortar cubes were prepared during the construction of each wall specimen. Three mortar cubes were sampled from each six batches of mortar used in the wall construction. The eighteen mortar cubes had a side length dimension of 50 mm. The mortar cubes were carefully cast in the mortar cubes mould and covered with plastic for three days. At this point the mortar cubes were transported to the BCIT Materials Lab and set in a moist curing tub at room temperature 20°C and 100% humidity as described in ASTM Standard C684, Procedure A. The preparation and storage of the mortar cubes conformed to the requirements set out in CSA Standard A179-04. Figure 3-15 displays a mortar cube being tested.
Mortar cube compression testing was performed in accordance with CSA Standard A179-04 when the mortar cubes had an age of 30 days. The tests were performed using the Forney LT-704-3 Universal Testing Machine in the BCIT Materials Laboratory. In order to ensure that the cubes were evenly loaded, a file and paper towel were used to smooth and clean the load application surfaces. Table 3-5 summarizes the results of the mortar flow tests after mixing and the mortar cube compressive strength test results are displayed in Table 3-6. The mean compressive strength was determined to be 12.4 MPa, slightly less than the minimum value of 12.5 MPa required for Type S mortar at 28 days but was nonetheless deemed adequate.

Table 3-5  Mortar cube compressive strength summary

<table>
<thead>
<tr>
<th>Mortar Cube No.</th>
<th>Mass (g)</th>
<th>Volume (cm³)</th>
<th>Density (g/cm³)</th>
<th>Compression Area (cm²)</th>
<th>Applied Load (kN)</th>
<th>Compression Strength (MPa)</th>
</tr>
</thead>
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<td>2660.46</td>
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<td>30</td>
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<td>39</td>
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<td>134.68</td>
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Mean: 32.3 MPa  
Standard Deviation: 3.9 MPa  
Coefficient of Variation: 12.1 %
Table 3-6  Mortar flow test results

<table>
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<th>Flow Test No.</th>
<th>Mortar Flow (mm)</th>
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</tr>
<tr>
<td>3-4</td>
<td>180.0</td>
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</tbody>
</table>

Mean: 177.9 mm  
Standard Deviation: 9.7 mm  
Coefficient of Variation: 5.5 %

3.3.3.5  Steel Reinforcement

Harris Rebar Ltd., a local steel reinforcement supplier, supplied all of the steel reinforcement used in the wall specimens. All steel reinforcement came from the same heat and was of Grade 400 deformed reinforcing steel. These were in lengths of 4400 mm, 700 mm, 3000 mm, and 2500 mm for the vertical wall reinforcement (both 15M and 10M), vertical dowels (10M), longitudinal footing reinforcement (15M), and horizontal wall reinforcement (10M) respectively. 15M and 10M reinforcing bars refer to 15 and 10 mm diameter steel bars respectively. There were also 34 transverse 15M reinforcing loops used for each footing. To determine the tensile stress and strain properties of the steel, there were five 500 mm long coupons cut from a single extra 15M and 10M 4400 mm length of reinforcement and again, came from the same heat. This was done prior to construction and the samples were then transported to the BCIT Structures Laboratory for testing.
The tensile strength tests were performed using the Baldwin Weidemann BTE-120 Universal Testing Machine. Each reinforcing bar sample was prepared according to CSA-G30.18-M92 (R2002) and ASTM A370, Section A9.3. The loading rate was applied according to the ASTM A370 standard, Section 7.4. An Epsilon testing instrument, a VL7A testing instrument, and a Kyowa electronic strain gauge were used to measure strains during the test. In case of misreadings from the strain gauges there was also a 200 mm range linear potentiometer set up as well. The Kyowa electronic strain gauge was glued to the specimen and provided strain - deformation data directly to the DAQ6 data acquisition program. The loading data output was then obtained directly from the testing machine. Figure 3-16 shows the tensile strength setup including the strain gauges and the backup linear potentiometer and Table 3-7 summarizes the tensile test results.

![Tensile strength test setup at the BCIT Structures Laboratory](image)

**Figure 3-16**  Tensile strength test setup at the BCIT Structures Laboratory
Table 3-7  Steel reinforcement properties

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>Area (mm$^2$)</th>
<th>Yield Load (kN)</th>
<th>Yield Stress (MPa)</th>
<th>Yield Strain (m/m)</th>
<th>Modulus of Elasticity (GPa)</th>
<th>Ultimate Load (kN)</th>
<th>Ultimate Stress (MPa)</th>
<th>Fail Load (kN)</th>
<th>Fail Stress (MPa)</th>
<th>Elongation (%)</th>
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<td>748</td>
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<td>55.1</td>
<td>627</td>
<td>11.1</td>
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</table>

10M Reinforcement Property Summary

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<th>Mean:</th>
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<th>506</th>
<th>0.00303</th>
<th>232</th>
<th>67</th>
<th>762</th>
<th>58</th>
<th>663</th>
<th>12.0</th>
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<tr>
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<td>0.6</td>
<td>11.3</td>
<td>0.0</td>
<td>5.5</td>
<td>0.7</td>
<td>13.4</td>
<td>6.3</td>
<td>78.1</td>
<td>1.5</td>
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<tr>
<td>Coefficient of Variation:</td>
<td>1.3</td>
<td>2.2</td>
<td>5.2</td>
<td>2.4</td>
<td>1.0</td>
<td>1.8</td>
<td>10.9</td>
<td>11.8</td>
<td>12.5</td>
</tr>
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</table>

15M Reinforcement Property Summary

<table>
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<tr>
<th>Mean:</th>
<th>84</th>
<th>504</th>
<th>0.00298</th>
<th>212</th>
<th>137</th>
<th>825</th>
<th>125</th>
<th>755</th>
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<tr>
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<td>6.9</td>
<td>0.0</td>
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<td>3.6</td>
<td>8.5</td>
<td>57.5</td>
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<tr>
<td>Coefficient of Variation:</td>
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<td>1.4</td>
<td>12.7</td>
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<td>1.3</td>
<td>0.4</td>
<td>6.8</td>
<td>7.6</td>
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</tbody>
</table>

The reinforcing steel satisfied the minimum requirements set out in CSA G30.18-M92. This standard stipulates that all steel reinforcement must demonstrate a minimum yield and ultimate strength of at least 400 MPa and 540 MPa respectively and that the minimum elongation at ultimate is at least 13%. The stress-strain relationships for each size of reinforcing bars is based on the recordings from the strain gauges. These relationships could be established from the combination of data from the strain gauges and the testing machine load cell.

It should be noted that there were many slips which occurred during the tensile tests despite effort to prevent this. This was deemed satisfactory after scaling out the additional strain which occurred with each slip however. It is also noteworthy that some of the strain gauges could not obtain the maximum ultimate strength as they became debonded before the strain at ultimate
could be reached. These figures also showed that the maximum average elongation at rupture for
the test specimens varied between 21% and 39%. The results indicated that the onset of strain
hardening occurred at 2.3% (an average value). This is less than the maximum tensile strain
which the reinforcement in the exterior of the wall specimens were designed to reach (8%). The
average yield strain was approximately 0.3% and the average yield strength was 505 MPa.

3.3.3.6 Footing Concrete

The concrete used in the wall specimen footings was supplied Ocean Concrete Ltd. and was 30
MPa GU Portland Cement Concrete (no AEA added). The concrete was supplied to the UBC
Structures Laboratory by means of a concrete ready-mix truck. The concrete was poured directly
into the footings and at this time there were ten concrete cylinders prepared. The slump at this
point was measured to 117 mm giving appropriate workability for the pouring. Both of the
footings were poured at once.

The concrete compressive strength of the footings was determined by testing the concrete
cylinders. There were standard concrete cylinders (200 mm height and 100 mm diameter) and
were prepared by filling ten 200 mm high, 100 mm diameter standard plastic moulds. The plastic
moulds were greased beforehand to aid in specimen removal later on.

The moulds were covered and left in the UBC Structures Laboratory next to the footings to best
represent their environment for the first three days. At this point the cylinders were transported to
the UBC Materials Laboratory to be put into the moist curing tank at room temperature of 20°C
and 100% humidity. The cylinders were then transported to the BCIT Concrete Laboratory at 28
days where their surfaces ground smooth using the BCIT Materials Laboratory's concrete
cylinder end grinder. This was done to ensure uniform distribution of the compressive load.
Additionally, two rubber pads were placed at the top and bottom of the grout prisms during the
testing. The compressive strength was determined in accordance with CSA Standard A179-04.
Due to an unanticipated delay, the testing was conducted at 43 days from casting. The testing
was done at the BCIT Concrete Laboratory using the standard Forney LT-704-3 Universal
Testing Machine. The failure of the concrete cylinders was also characterized by vertical shear
failure in all cases. The average footing concrete compressive strength was 45.2 MPa (based on
10 cylinders). The full results of the concrete cylinder compressive strength test are summarized in Table 3-8.

### Table 3-8  Foundation concrete cylinder compressive strength summary

<table>
<thead>
<tr>
<th>Cylinder No.</th>
<th>Height (mm)</th>
<th>Mass (kg)</th>
<th>Density (kN/m$^3$)</th>
<th>Load (kip)</th>
<th>Load (kN)</th>
<th>Compressive Strength (MPa)</th>
<th>Age (days)</th>
</tr>
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<td>3.6</td>
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<td>74.522</td>
<td>331</td>
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<td>24.22</td>
<td>86.605</td>
<td>385</td>
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<td>75.727</td>
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<tr>
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<td>1-10</td>
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<td>24.13</td>
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<td>350</td>
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<td>43</td>
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</tbody>
</table>

Mean: 43.4 MPa
Standard Deviation: 6.4 MPa
Coefficient of Variation: 14.7 %

### 3.4  Test Setup

To study the out-of-plane instability failure mechanism described in Chapter 1 and 2, a setup which could transfer both axial compression and flexural loads to the wall specimens during testing was necessary. This required the test setup to be custom designed for this study as shown in Figure 3-17. The test setup was modified for each specimen W1 and W2, the only difference being the absence of the two vertical actuators supplying the additional axial load for the second specimen, W2. Due to the flexural nature of the out-of-plane instability failure mechanism, the test setup also required the capability to apply the axial and flexural loads while also minimizing the shear loads on the wall specimens. Furthermore, it was desired that the wall specimens were
to have the laterally applied loads transferred uniformly over the length of the wall specimens rather than through a two bearing connections at their top corners. The imposed lateral loads and overturning moments were also to be increased and decreased proportionally based on the NBCC 2005 inverted triangular loading distribution to maintain the proper moment to shear ratio so that the governing failure mode would be flexural. In order to impose this type of loading on the wall specimens the test apparatus needed to incorporate a number of important design aspects to be discussed in detail in this section.

All steel components of test setup were built and assembled with the guidance and advice of the trained laboratory technicians from the UBC Structures Laboratory. Prior to the test there were a wide variety of checks on the orientation of all test setup components to ensure proper orientation and stability. All components of the test setup were connected as specified in design drawings using the proper equipment to complete the task.

The test setup also required to be incorporate instrumentation to record the displacements, loads, rotations and strains in order to capture the overall behaviour of the wall specimens under the imposed loading. This instrumentation had the requirement to be installed such that the recordings could be made over the height of the specimens and throughout the duration of the tests. A detailed description of the instrumentation used in the test setup will also be discussed later in this chapter.

The objective of the test setup was to simulate the effect of seismic loads on the lower portion of a 9 m high wall in a commercially loaded, three-storey building (referred to herein as “simulated wall”) for W1 and to provide a "worst-case" scenario for W2 which would be characterized by a tall wall in a one-story warehouse. The vertical compression loading was held constant at 660 kN for W1 (corresponding to an axial compressive stress of 0.034f'm) and was non-existent for W2. Lateral load on the specimens was applied through the horizontal actuator in combination with the net horizontal load from the sum of the two vertical actuators (for W1 only). The overturning moment representing the loads in the upper stories of the simulated wall was created by combined the effects of lateral and vertical loads of the actuators for W1 and only the horizontal actuator load for W2. The loads in the vertical actuators were slaved to the load from the horizontal actuator by a linear relationship derived from equilibrium equations relating the simulated wall to the wall specimen (for W1 only). This relation is demonstrated in Figure 3-18.
Figure 3-17  Test Setup for W1 (W2 identical but without use of Actuators 2 and 3)
Figure 3-18  Applied lateral loads on the simulated wall and equivalent loads on W1
3.4.1 Test Apparatus - W1 Actuators

The design of the test apparatus for the W1 test took into consideration a number of factors which were imperative to the objectives of this experimental study. The test apparatus required the capability to apply both axial loads representing the gravity loads applied on top of the wall specimens as well as significant flexural loads to induce a high overturning bending moment at the base of the wall specimen. Complicating the matter was the fact that the wall specimen must be loaded so as to force a flexural failure mechanism to govern over a shear mechanism. As noted in previous sections, the desired out-of-plane failure mechanism is contingent on developing a flexural failure resulting from in-plane loading. Therefore, if a shear failure preempted the out-of-plane instability the test would be null. As a result, the test setup was designed to incorporate one horizontal and two vertical actuators working in conjunction. The actuators were servo-controlled and were of MTS design and manufacture. The horizontal actuator was a MTS Model 244.51, with a maximum capacity in tension of 1000 kN and a stroke of ±508 mm. The two vertical actuators were MTS Model 201.60's and were not actually perfectly vertical but were inclined at an approximately 13° angle with regard to vertical with a load capacity of 645 kN and stroke of ±305 mm.

This incline of the vertical actuators was incorporated into the test apparatus design so that the vertical loads applied by the actuators could be imposed directly upon the wall specimen rather than from a cantilevered loading beam. This enabled more uniform load transfer from the loading beam to wall specimen as well as reducing the size of the loading beam channels and hence cost which was a very important parameter in this study. If Actuators 2 and 3 were mounted exactly vertically, the loads would have to be transferred to the loading beam which cantilevered off the ends of the wall specimen. In this situation, there would be a formation of large bending moments and deflections occurring within the loading beam due to the high amounts of compression load applied upon it. This would not only increase the necessary size of the loading beam, but also cause the stress to concentrate on the corners of the wall specimens and especially the vulnerable face-shells on the wall ends. By using a loading beam which did not cantilever off the ends of the wall specimens the load was transferred directly on top of the wall through both bearing and through the numerous shear rods. The load path in this case would be much more predictable. Of course, numerous adjustments and post-processing tasks were
needed to be made for the loading protocol and the data acquisition respectively due to this arrangement of the vertical actuators.

These two actuators were located on either end of the wall specimens. (The horizontal actuator will hereupon be referred to as Actuator 1 and the vertical actuators as Actuators 2 and 3 for the South and North end of the wall specimens respectively. Together these three actuators could produce both the necessary overturning moment and axial gravity loads. If the vertical actuators acted to only supply constant axial gravity loads vertical actuators and the horizontal actuator providing the only flexural force, the calculated required loads could not have been achieved.

The advantage of using the three actuators together in synchronization was that the applied shear load imposed upon the wall specimens would be reduced. This is because each actuator provides a component to the overturning moment at the base of the wall specimen. This of course enables a behavioural preference for the wall specimens to fail due to flexure rather than shear. Providing the overturning moment at the wall base through the use of only the Actuator 1 requires high loads to be induced by this actuator. The high shear loads formed from this loading require large amounts of reinforcement to be located in the wall specimens due to their relatively small block width. Despite this fact, the test on W2 provided the loading in this manner in order to observe the effect of axial loads on the out-of-plane instability failure mechanism. As anticipated, issues arose regarding sliding at the wall base during this test. Nonetheless, the factors of safety used in the original specimen design provided enough buffer for W2 perform adequately during the testing.

As already stated previously, the objective of the test setups was to simulate the effect of seismic loads on two different types of walls typically encountered in Canadian practice. For W1, the wall specimen was designed to represent the bottom portion of a taller wall and thus the Actuators 2 and 3 were incorporated to represent both the gravity loads and the imposed overturning moments from the floors above. They had an advantageous side effect in that they also contributed to the total shear force from the floors above from the summation of the lateral components of their incline loads. These actuators therefore not only provided greater flexibility for the testing in attempting to force an out-of-plane instability failure mechanism, but also to model taller walls which could not have been physically tested due to the height limitation of the
UBC Structures Laboratory. Due to their dual function, together they were designed to apply a constant total axial load upon the wall specimen while also permitting the application of a load differential between the two actuators to generate an overturning moment at the top of the wall specimen.

Using simple equilibrium equations relating the simulated wall to the actual wall specimens it was possible to develop a linear relationship between the necessary loads required from the vertical Actuators 2 and 3. The idea was based on the principle that if Actuator 1 extended in one direction, one of the vertical actuators would release a portion of its load and the other vertical actuator would increase its load by an equal portion in order to maintain a constant axial load on the specimen. For example as Actuator 1 applied load in the North direction, Actuator 3 would increase its load and Actuator 2 would decrease its load by an equal amount. This produced an additional overturning moment at the base of the wall specimen which would be combined with that produced by the lateral load from Actuator 1. The lateral components of these changes in load caused an additional shear component in the direction of the displacement of Actuator 1 which was taken into account in the post-processing of the data. This relationship may be summed up in the following two equations:

\[ A_2 = P_a - 0.7A_1 \]  

\[ A_3 = P_a + 0.7A_1 \]

As shown in Figure 3-17, the Actuators 2 and 3 were installed on an incline of 13° from vertical at the beginning of the test (this angle would vary slightly as imposed lateral displacements at the top of the wall specimen increased) and the effects of this were taken into account in Equations 1 and 2 above. Equation 1 describes the load control on Actuator 2 in terms of the initial constant axial load, \( P_a \), and the variable load in Actuator 1, \( A_1 \). Equation 2 describes the load control on Actuator 3 with the same variables assigned in Equation 1.

The equilibrium relationship which determined the relative magnitudes of load required from each actuator was used to define the slaving matrix which linked the load commands in Actuators 2 and 3 to the load feedback from Actuator 1 to the loads induced by the load cell of
the horizontal actuator, Actuator 1. The relationship used for the slaving matrix is illustrated below in Figure 3-27.

Figure 3-19  Actuator load relationship in slaving matrix for loading in the North (top) and South directions (bottom). Yellow lines indicate the constant tensile axial loads, red lines indicate the additional tensile axial load to produce additional overturning moment, and blue lines represent the compressive/tensile lateral load.
This necessarily meant that Actuators 2 and 3 must be force-controlled. This type of control is generally avoided in most testing circumstances due to the inherent danger when strength degradation begins to occur within the test specimens. However, due to the fact that Actuator 1 was displacement-controlled, Actuators 2 and 3 could largely be controlled through the lateral displacement command. In the case of strength degradation in the wall specimen there would be a reduction in the lateral load applied by Actuator 1 and its load cell. This reduced load would then result in reduced loads in Actuators 2 and 3. As long as the lateral load capacity reduced at a similar rate as the axial load capacity the control of the test could be well maintained during strength degradation.

Since Actuators 2 and 3 would always be in tension for the constant axial load, their connection to the loading beam could be simplified. Actuators 2 and 3 were connected to the loading beam via two 36 mm diameter Dywidag bars. These bars ran from a plate connected to the actuator to a plate atop the loading beam. There was a Dywidag domed nut sitting within a recessed plate to provide the top connection with lateral displacement capabilities and what was an essentially pinned connection. The actuators also had another of these pinned connections at their base which was connected to a plate which in-turn was connection to one of the vertical actuators. Additionally, a built in swivel included in the standard MTS Model 201.60 provided more flexibility to movement during the testing. The swivel was anchored to the UBC Structures Laboratory’s strong floor by means of two 2” bolts.

Actuator 1 also utilized double-pin connections. The standard MTS Model 244.51 also had built-in swivel connections at both ends of the actuator. These were bolted to the UBC Structures Laboratory’s strong wall and the loading beam by means of a single 44 mm diameter Dywidag bar and four 1¼” high-strength bolts respectively.

All three of the actuators used in the test apparatus were compatible with one another and with the data acquisition system allowing for the control to be in the master and slave fashion which was critical to the loading protocol. Actuator 1 was mounted near the top of the UBC Structures Lab’s strong wall and Actuators 2 and 3 were mounted on either end of the wall specimen to the UBC Structures Lab’s strong floor. Figure 3-19 displays the mounting of the three actuators of the test setup.
This type of slaved loading had not been done in the UBC Structures Laboratory prior to these tests and a timber shear wall was built to act as a "dummy wall" to ensure all components of the testing behaved as expected. The timber was built to the same cross-sectional dimensions as the actual wall specimens but to half the height to reduce the size of the timber members. The dummy tests were run numerous times at relatively low displacement levels before it was determined the loading protocol could be run effectively. Figure 3-29 displays the test setup for the dummy wall tests.
3.4.2 Test Apparatus - W2 Actuators

The design of the test setup for the W2 test was considerably simpler than that for W1. The physical setup was essentially the same except for the removal of applied load from Actuators 2 and 3. The wall specimen, W2, represented only the bottom storey of a warehouse structure and thus received negligible applied axial loads at its top. In addition to lack of axial load, there was no reason for additional overturning moments arising from the mass of stories above. This permitted the entire test to be run with a single actuator, Actuator 1 applying lateral load to create the necessary overturning moment at the base. This of course led to a tendency for base slip and sliding which will be discussed in the next chapter. Despite this behaviour, the governing failure mode of W2 was still considered to be flexural. This was in part due to the factor of safety of 2.5
used in the original design to prevent against a shear failure mode and force the occurrence of a flexural failure mode instead.

3.4.3 Test Apparatus – Loading Beam

The loading beam employed for the testing was designed for a number of important connections and was composed of a number of steel section components. As shown in Figure 3-20, the loading beam consisted of two 500 mm deep channel sections facing away from one another and five large steel plates connecting the channels together, three located on top of the loading beam and one located at each end. In addition to these connection plates, each channel had two vertical stiffeners welded on each end below the connections for Actuators 2 and 3 and three stiffeners welded along their length. These stiffeners were $3 \times \frac{1}{2}''$ (76.2 x 12.7 mm) flat bar plates cut to fit between the flanges of the channel sections. A 5 mm fillet weld ran along both sides of each stiffener. There was also one vertical capping plate on the end of each channel. Each plate was 1'' (25.4 mm) thick and welded directly to the end face of the channel using both notch and fillet welds. The plates were complete with a drilled hole pattern to permit connection with the larger end connection plate. The end of the beam which supports the Actuator 1 connection had its vertical capping plates reinforced with four stiffeners to minimize warping and deformations from occurring within the plate.
The channels were spaced 200 mm apart so that they sat on either side of the wall specimens and thus effectively sandwiched it. Because the concrete block units had a width of 190 mm at the top of the wall specimens, there was a gap of 5 mm on either side of specimens hypothetically. This play allowed the accommodation for variances in the masonry plumbness which later proved to be necessary during the mounting of the loading beam onto the wall specimens.

The two vertical end connection plates were of different dimensions. One connection plate, on the South end of the wall specimen, was used to connect the channels together as well as providing the connection for Actuator 1. This plate had dimensions of 18 x 18" (457.2 x 457.2 mm) and was 1" (25.4 mm) thick. On the North end of the channels, opposite to Actuator 1 there was a 500 mm x 15" (381 mm) dimensioned 1" (25.4 mm) plate attached to connect together the channels at this end of the loading beam.

In addition to the vertical connection plates, there were three connection plates oriented horizontally across the top of the channels. These plates acted to provide a large portion of the bearing for the axial gravity loads to the top of the wall specimens. The plates connecting the
channels at the top consisted of one 32 x 15 x 1½" (812.8 x 381 x 38.1 mm) steel plate on each end of the loading beam and one 32 x 15 x 1" (812.8 x 381 x 25.4 mm) steel plate located at mid-length. These plates were bolted directly through recessed holes drilled through the channel flanges. This provided more play than a welded connection when the loading beam was put into place over the top of the wall specimen and during removal. This process was difficult due to the height restrictions within the UBC Structures Laboratory. An added benefit of this arrangement was to allow the steel sections to be reused later for further testing on differently shaped wall specimens. The end plates were bolted with four 1" (25.4 mm) A490 flat head socket cap screws each and the middle plate was bolted with four 1" A325 regular hexagonal bolts. The flat head socket cap screws were used to provide a flat surface on top of the end plates. This permitted the recessed Dywidag plates to move laterally during the testing. Acting as washers, the recessed plates for the Dywidag bars sat on top of 55 mm holes complete with 45° beveled edges drilled through the end plates. These holes were deliberately drilled too large to accommodate the inclined 36 mm Dywidag bars. This effectively provided each actuator with a pin-ended connection as can be seen in Figure 3-21. These same holes had a dual purpose and that was to act as the top restraint for the specimen transportation system.

Figure 3-23  Pin connection of vertical actuators to loading beam
It was critical that there were no tensile loads induced within the wall specimens during transportation from the construction location to the testing location. These loads could have created premature cracking of the specimen’s mortar joints and could perhaps have altered the results or observations of the wall specimens during the testing. In order to prevent this from occurring, the wall specimens were post-tensioned before lifting. This was done by connecting six 1” (25.4 mm) steel threaded rods from holes drilled within the three top connection plates of the loading beam and the six coupler holes embedded within the wall specimen foundation as described earlier. As shown in Figure 3-22, the three sets of rods were located on both ends and in the center of each side of the wall specimens. Once in place these rods were sufficiently tensioned such that as the loading beam was lifted, the wall specimens would remain in compression. The post-tensioning was done by means of torquing nuts fastened to the top of the rods above the loading beam connection. Careful attention was paid when performing this post-tensioning to prevent different compressive loads and possible instability from forming. This was done by slowly tightening the torque nuts on each of the rods in a sequence which allowed the specimens to be compressed over their entire cross-section nearly uniformly. During the post-tensioning, rubber pads were placed beneath the bearing plates of the loading beam in order to prevent damage to the bearing surfaces of the wall specimens. The wall specimen and loading beam were moved as a unit by the UBC Structures Laboratory 10-ton rail crane via two sling connections strongly welded to the top connection plates of the loading beam. After and during the moving of each specimen into place, the mortar joints were carefully inspected to ensure that no cracks formed during the lifting and transportation. No evidence of cracking was found in any of the wall specimens after their placement into the test setup.
Two out-of-plane supports prevented out-of-plane movement of the top of the wall specimens during the testing. These out-of-plane supports consisted of a solid cylindrical steel strut rod complete with end eyes which permitted an adjustable length. Each of these supports were attached to the top of the loading beam on one end and to one of the support columns on the other end as shown in Figure 3-23. Each connection was provided by means of a steel cylindrical pin. The pin on the end connected to the support column was braced by a swivel head. The pin on the end connected to the loading beam was braced as a vertical dowel extending from a ½" (12.7 mm) steel plate welded to the top of the loading beam. This rigid pin connection of course meant that there were would actually be out-of-plane displacements imposed by the out-of-plane supports onto the wall specimens as the lateral in-plane displacements of the wall specimen increased. However, the length of the lever arm was specifically chosen so as to minimize this effect. A longer arm necessarily means that a given in-plane lateral displacement of the wall specimen produces a smaller rotation of the out-of-plane support. This in turn results in a smaller out-of-plane displacement imposed upon the wall specimen. The out-of-plane support length of
1020 mm was designed so that a 70 mm lateral in-plane displacement of the wall specimen would result in a negligible corresponding 2.5 mm out-of-plane displacement. See Figure 3-24 for the circular relationship between a given in-plane lateral displacement and its corresponding out-of-plane displacement. While the out-of-plane supports were designed to guide the wall specimens along their lateral in-plane displacement path for anticipated displacements, their relatively small diameter of 1¾" (44.5 mm) diameter meant that their load carrying capacity was limited. For this reason, two 3200 lb capacity slings were attached from the support columns to the loading beam. The sling lengths were long enough as to not interfere with the tests but short enough to prevent a potentially dangerous collapse of the wall specimen.

Figure 3-25   Rigid pin-connection between loading beam and out-of-plane support
The connection between the loading beam and the wall specimen was established through 22 - \( \frac{3}{4}'' \) (19.1 mm) steel threaded rods extending between the two channels of the load beam. This connection provided not only the lateral load transfer but also a large portion of the vertical loading as well. This type of connection was used so as to simulate the transmission of lateral seismic forces during an earthquake to a RMSW by applying a uniform lateral load along the length of the wall specimen. This would represent the loads transferred through the wall specimen and slab reinforcement rather than a single concentrated bearing point load at one of the top corners of the wall specimens. Naturally a biproduct of this type of loading is that the load transfer mechanism is much more clear than the potential load paths arising from a bearing connection. The test setup was actually designed to prevent any lateral transfer of load through bearing to the wall specimens and there was a 2½" (63.5 mm) gap left between the loading beam end plates and the specimen to ensure this goal was achieved throughout the testing.

Figure 3-26  Out-of-plane versus in-plane lateral displacement relationship

To create this steel threaded rod connection, there were 22 - 1" (25.4 mm) diameter holes drilled completely through the wall specimens through existing 28.5 mm diameter holes in each loading
beam channel after the loading beam was in place. The number and size of the holes were determined by designing the rods for the maximum biaxial shear loads they would receive and taking into account the size of the existing holes in the loading beam channels. The location of the holes was determined through analysis to prevent local failure at the top of the loading beam and also taking into account the existing holes in the loading beam channels. A Hilti concrete hammer drill was used for the drilling. A ½" (12.7 mm) diameter pilot hole was drilled first from each side of the wall specimen in an attempt to keep the through hole level. This was followed by the 1" (25.4 mm) diameter drill bit and hole. At this point, each hole was air blown out and brushed clean to the Hilti specifications.

A custom built bushing was then hammered into each hole of the loading beam in order to remove any play in the hole for the ¾" (19.1 mm) diameter threaded rods. This play would have caused very detrimental effects during the lateral cycling during the testing as the gap would cause a large discrepancy between the actuator commands and the predetermined loading protocol. Although the actuator displacements may have matched the loading protocol displacement levels, the actual wall specimen displacements would be lagging behind the size of the gap. The bushing outside diameter was slightly less than 1" (25.4 mm) and the inner diameter was slightly more than ¾". One end of the hole was then filled with Hilti HIT-RE 500 Epoxy Adhesive as per the Hilti instructions and rod was inserted into this end through the corresponding hole and bushing in the loading beam channel. The epoxy gun was then transferred to the other, second side of the hole and a steady, thick bead of epoxy was laid in front of the rod as it was slowly inserted into the hole from the first side of the hole. Careful attention was given to ensure that no voids existed between the threaded rod and the inner hole surfaces. The rod was then fully inserted when it passed through the bushing inserted into the hole on the second side of the loading beam. A washer, split ring lock-washer, and nut were then installed on the threaded rod next to each bushing and tightened to minimize the gap between the wall specimen and the inner face of the load beam. This process was repeated for each of the 22 hole locations.
3.4.4 Strong Wall and Strong Floor Mountings

The "strong wall" and "strong floor" of the UBC Structures Laboratory were essential components to the testing of the wall specimens. Each of these components consisted of heavily reinforced, post-tensioned concrete members and provided a practically infinitely stiff base from which to push and pull. The actuators, support columns, and the wall specimens themselves were bolted to either the strong wall or strong floor. This discussion will briefly describe these types of connections.

![Diagram of Actuator 1 connection to UBC Structures Lab strong wall]

**Figure 3-27 Actuator 1 connection to UBC Structures Lab strong wall**

Actuator 1 was connected through the 1 m thick UBC Structures Laboratory post-tensioned strong wall. The actuator's base swivel head was bolted to a custom-built connection system, as shown in Figure 3-25, by means of 4 - 1½" (38.1 mm) diameter high-strength bolts. This consisted of two 18 x 6 x 4" (457.2 x 152.4 x 101.6 mm) steel mounts welded to a 32 x 15 x 2" (812.8 x 381 x 50.8 mm) structural steel plate by means of a 15 mm wide complete joint penetration weld. The plate had a 45 mm diameter hole drilled between the steel mounts and from which a 44 mm diameter high-strength Dywidag rod was inserted. This rod protruded to the other end of strong wall via one of the cast holes within the strong wall. The Dywidag rod was fastened in place by high-strength Dywidag nuts on each end with the Dywidag recommended rod extensions taken into consideration. The side of the strong wall opposite the test setup had a
12 x 12 x 1" (304.8 x 304.8 x 25.4 mm) plate to act as a washer and also to spread the actuator tensile loads from the nut across a larger area of the strong wall to minimize potential damage.

Actuators 2 and 3 were connected to the strong floor of UBC Structures Laboratory in a similar manner as shown in Figure 3-26. The actuator swivel head of each actuator was bolted to a 32 x 15 x 2" (812.8 x 381 x 50.8 mm) structural steel plate by 4 - 1¼" (38.1 mm) high-strength bolts. These plates were then fastened to the strong floor by two 44 mm diameter Dywidag rods which extended through two 45 mm diameter drilled holes in each plate to cast holes within the strong floor. In the basement beneath the 12" (304.8 mm) thick heavily reinforced strong floor, a 12 x 12 x 1" (304.8 x 304.8 x 25.4 mm) plate was again used as a washer and to spread each of the actuator's tensile loads from the nut across a larger area of the strong floor ceiling. A Dywidag nut was tightened on each end of each rod to provide a firm actuator connection to the strong floor.

Figure 3-28   Vertical actuator connections to UBC Structures Lab strong floor: above strong floor connection (left) and below strong floor connection (right)

There were four support columns connected to the strong floor as well. Each of these columns were standard components belonging to the UBC Structures Laboratory. The columns were W310x158 steel sections welded to a 32 x 15 x 3" (812.8 x 381 x 76.2 mm) structural steel plate. Two of these columns were located on either side Actuator 1 to support it during specimen placement and removal. The other two specimens supported much of the testing instrumentation as well as the out-of-plane supports for the wall specimens. These columns were supported against lateral displacements in the out-of-plane direction relative to the wall specimens. These
lateral displacements were produced by the reactions of the out-of-plane supports from the in-
plane loading imposed on the wall specimens. It was of critical concern that the instrumentation
attached to these columns remain stationary throughout the testing. This support was by the
means of a HSS 203x203x9.5 steel section connecting the top of the of the column to connection
pads attached to the strong wall by two 1" (25.4 mm) diameter threaded rods extending through
cast holes in the strong wall. Since this connection would only be in compression, the HSS
section was hammered into place and a 3" (76.2 mm) plywood mount was used to connect the
steel section to the threaded rods. Despite this connection being deemed adequate, it was
monitored during the tests for any displacement of which there fortunately was none. All of the
columns were connected to the strong floor by two 2" (50.8 mm) diameter steel threaded rods
complete with the appropriate nuts and washers. The washers on the bottom side of the rods were
placed beneath a 12 x 12 x 1" (304.8 x 304.8 x 25.4 mm) plate to prevent damage to the strong
floor during tensioning.

Each wall specimen was securely connected to the strong floor in a similar manner to the
connection of Actuators 2 and 3 as shown in Figure 3-27. There were eight 2" (50.8 mm) holes
cast into the footing of each specimen at the same spacing as the hole pattern of the strong floor.
When the wall specimen was moved into the test setup, a 38 mm diameter Dywidag rod was
inserted into each of these eight holes. The area around each of the footing holes was capped
with concrete to provide an even and flat bearing surface for a 7 x 7 x 1½" (177.8 x 177.8 x 12.7
mm) washer plate. A second plate of similar size was inserted on the bottom end of each rod.
These plates were used to spread the forces from the tensioning of the rods across a larger
surface. The rods were tensioned by tightening Dywidag nuts on each end of rods to a force
which would prevent sliding of the footings during testing according to the Coulomb friction
theory. To aid in preventing sliding during the tests, one large cylindrically coned bushing was
inserted at the Northwest and Southeast corners of the footings. The bushings were hammered
into place to remove any play in the holes connecting the footing to the strong floor.
A single instrumentation scheme was used for each of the tests W1 and W2 with only minor changes in the locations of some of the individual instruments for W2. The layout of the instrumentation was developed such that all necessary information was obtained using the optimal amount of data. Due to the large size of the wall specimens and the equally large number of parameters desired to be measured during the testing, there was a correspondingly large number of instruments incorporated into the test setup. Predictions of approximate displacements, strains and rotations were calculated through a theoretical analysis using design programs and hand calculations based on established concepts of mechanics and materials. These predictions were necessarily conservative so as to ensure the instrumentation would be able to sustain the demands imposed upon it. Using these conservative predictions it was possible to select appropriate instrumentation for the testing in order to measure the desired parameters.

Displacements, loads, rotations and strains were all monitored during testing by a large number of channels (53 for the W1 test and 54 for the W2 test feeding into a data acquisition system which took recordings at 0.1 second intervals. In the test on W1, a total of 21 string potentiometers were used to measure vertical in-plane, horizontal out-of-plane, and diagonally...
in-plane deformations of the W1. For W2 there were 22 string potentiometers used. The configurations of these instruments created an effective strain rosette required to distinguish between flexural and shear deformations. Figure 3-28 demonstrates a cluster of string potentiometers forming a part of the strain rosette.

![Image](image.jpg)

**Figure 3-30  Cluster of string potentiometers forming a part of the strain rosette measuring vertical and diagonal displacements**

The vertical in-plane instruments were used to determine the axial deformation at each end of the wall specimens both over the total height and over the two vertical halves of the plastic hinge region. At the North end of the wall, two of these vertical instruments were setup on both the East and West faces to detect relative displacement over the plastic hinge region during a potential out-of-plane instability mechanism. The horizontally out-of-plane instruments were installed to measure the progressive out-of-plane displacements associated with this mechanism. The mount locations for the South end lower out-of-plane displacement readings are shown in Figure 3-31.
Complementing these string potentiometers were 16 linear rod potentiometers measuring a number of parameters; out-of-plane displacements in the plastic hinge region, in-plane lateral displacements, wall uplift at the wall ends, base slip/sliding, relative displacement at the interface between the loading beam and the wall specimens, and finally the lateral movement of the support columns due to possible loading from the out-of-plane supports. There were also two inclinometers positioned at South end of the wall specimens to measure the rotation at an anticipated location of out-of-plane instability. The two inclinometers are shown in Figure 3-32 as well as one of the base uplift linear potentiometer instruments.
Figure 3-32  Instrumentation at wall specimen South end: a) inclinometers recording end zone rotations and b) vertical linear potentiometer detecting wall base uplift

Internally, there were four strain gauges applied to the vertical reinforcing steel at each of the bottom corners of the wall specimen prior to construction. These were placed at the second and third bed joints from the footing as shown with the rest of the instrumentation in Figure 3-25. Figure 3-24. The strain gauges were used to monitor the extent of yielding in this region. This was a critical for the development of the out-of-plane instability mechanism. All actuators also produced outputs of their axial loads and displacements. Table 3-9 displays the instrumentation numbering and descriptions as well as maximum stroke and resolution of each instrument used in the testing. Figures 3-35 and 3-36 illustrates the locations of these instruments.
Figure 3-33 Strain gauge locations on reinforcing bars at mortar joints on each end of wall specimens

As the in-plane displacements were recorded at the top of the wall specimen both by the displacement feedback from Actuator 1 and the displacement readings recorded by string potentiometer directly below the connection point of Actuator 1. This was done to check the displacements for agreement. Figure 3-34 shows the location of the string potentiometer.
Figure 3-34  Location of string potentiometer used to check lateral displacement at the top of the wall specimen

Table 3-9  Instrumentation summary

<table>
<thead>
<tr>
<th>Instrument Identification Number</th>
<th>Instrument Description</th>
<th>Full Stroke &amp; Resolution</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Diagonal String Potentiometer</td>
<td>100 mm +/- 0.1 mm</td>
</tr>
<tr>
<td>2</td>
<td>South End Vertical String Potentiometer</td>
<td>50 mm +/- 0.5 mm</td>
</tr>
<tr>
<td>3</td>
<td>Out-of-Plane String Potentiometer</td>
<td>100 mm +/- 0.1 mm</td>
</tr>
<tr>
<td>4</td>
<td>Diagonal String Potentiometer</td>
<td>100 mm +/- 0.1 mm</td>
</tr>
<tr>
<td>5</td>
<td>North End Vertical String Potentiometer</td>
<td>50 mm +/- 0.1 mm</td>
</tr>
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<td>6</td>
<td>South End Vertical String Potentiometer</td>
<td>250 mm +/- 0.5 mm</td>
</tr>
<tr>
<td>7</td>
<td>Diagonal String Potentiometer</td>
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</tr>
<tr>
<td>8</td>
<td>North End Vertical String Potentiometer</td>
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</tr>
<tr>
<td>9</td>
<td>North End Vertical String Potentiometer</td>
<td>50 mm +/- 0.1 mm</td>
</tr>
<tr>
<td>10</td>
<td>North End Vertical String Potentiometer</td>
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</tr>
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</tr>
<tr>
<td>12</td>
<td>North End Vertical String Potentiometer</td>
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</tr>
<tr>
<td>13</td>
<td>North End Vertical String Potentiometer</td>
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</tr>
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</tr>
<tr>
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<td>50 mm +/- 0.1 mm</td>
</tr>
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<td>25 mm +/- 0.1 mm</td>
</tr>
<tr>
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<td>Instrument Description</td>
<td>Full Stroke &amp; Resolution</td>
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<td>----------------------------------------------</td>
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<td>North End Out-of-Plane String Potentiometer</td>
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<td>North End Strain Gauge</td>
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<td>38</td>
<td>South End Strain Gauge</td>
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<td>39</td>
<td>South End Strain Gauge</td>
<td>0.01 +/- 0.000001</td>
</tr>
<tr>
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<td>South End Strain Gauge</td>
<td>0.01 +/- 0.000001</td>
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<tr>
<td>41</td>
<td>South End Strain Gauge</td>
<td>0.01 +/- 0.000001</td>
</tr>
<tr>
<td>42</td>
<td>Wall Base Slippage Linear Potentiometer</td>
<td>10 mm +/- 0.1 mm</td>
</tr>
<tr>
<td>43</td>
<td>South End Inclinometer</td>
<td>90° +/- 0.1°</td>
</tr>
<tr>
<td>44</td>
<td>South End Inclinometer</td>
<td>90° +/- 0.1°</td>
</tr>
<tr>
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<td>Wall Base Slippage Linear Potentiometer</td>
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</tr>
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<td>46</td>
<td>Actuator 1 Displacement Feedback</td>
<td>1016 mm +/- 0.01 mm</td>
</tr>
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<td>Actuator 2 Displacement Feedback</td>
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<tr>
<td>48</td>
<td>Actuator 3 Displacement Feedback</td>
<td>610 mm +/- 0.01 mm</td>
</tr>
<tr>
<td>49</td>
<td>Actuator 1 Load Cell Feedback</td>
<td>1000 kN +/- 0.001 kN</td>
</tr>
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<td>50</td>
<td>Actuator 2 Load Cell Feedback</td>
<td>645 kN +/- 0.001 kN</td>
</tr>
<tr>
<td>51</td>
<td>Actuator 3 Load Cell Feedback</td>
<td>645 kN +/- 0.001 kN</td>
</tr>
<tr>
<td>52</td>
<td>Support Movement Linear Potentiometer</td>
<td>10 mm +/- 0.1 mm</td>
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<tr>
<td>53</td>
<td>Support Movement Linear Potentiometer</td>
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</tr>
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<td>In-Plane Linear Potentiometer</td>
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</tr>
<tr>
<td>55</td>
<td>In-Plane String Potentiometer</td>
<td>250 mm +/- 0.1 mm</td>
</tr>
</tbody>
</table>

The vertical and diagonal in-plane string potentiometers as well as a number of the in-plane linear potentiometers were applied directly to the face of the masonry using epoxy whereas the horizontal out-of-plane string potentiometers were attached to the support columns together with the corresponding linear potentiometers. All of these sensors were connected to the wall faces by wires to an epoxied eye-bolt mount. The in-plane linear potentiometers monitoring the lateral displacement of the wall specimens were attached to an instrumentation support frame. Two dial gauges were mounted on the lab strong floor to measure any horizontal slip of the concrete footing. The instrumentation in general was arranged to produce data that could effectively
calibrate a future analytical model to be implemented in the following stages of the research program.

To summarize, the following totals complete the number and tasks of the instruments used in the test setup; there were 7 instruments setup on the West face of the wall specimens, on both the North and South ends, to measure potential out-of-plane displacements over the height of the specimens. There were 3 instruments measuring the in-plane lateral displacement of the wall specimens over their height. Two instruments were in place to measure the overall vertical strain of the wall ends and 6 additional instruments to measure vertical strains of the plastic hinge region on the East and West faces of the walls specimens' North end. Four instruments measured the shear displacements of both the wall specimen's plastic hinge region as well as the rest of the specimen. Eight strain gauges measured the vertical strains in the exterior reinforcement at the wall toes. Two instruments were used to measured slip between the wall specimens and their footings as well as the between the wall specimens and the loading beam. Uplift between the footing and wall toes as well as vertical displacement between the loading beam and the wall specimen corners were both measured with 2 instruments each. Two inclinometers were used to measure potential rotations at the plastic hinge region as well. Finally there were 2 instruments employed to measure potential lateral displacement of the support columns in the out-of-plane direction with respect to the wall specimens. The overall scheme and location of sensors on the West and East faces of W1 and W2 are shown in Figure 3-35 and Figure 3-36 respectively.
Figure 3-35  Specimen instrumentation on West face
Figure 3-36  Specimen instrumentation on East face
3.4.6 Data Acquisition System

The data acquisition system used during the testing was essentially composed of three primary components; a Digital Equipment Station, a real-time data acquisition program called DASYLab, and an analog-to-digital converter. A signal conditioning unit was used to collect the recordings from all transducers, including the string and linear potentiometers, strain gauges, inclinometers, and the MTS actuator load cells and LVDTs. Each of these transducers was assigned a channel which was plotted in DASYLab.

Prior to the testing, all of these transducers and load cells were calibrated as per standard practice. The load cells were calibrated using the Baldwin-Tate-Emery Universal Testing Machine in the UBC Structures Laboratory. The transducers were calibrated using displacement gauges. During testing, the control system provided a real-time display of several desired plots using a number of different transducers. The plots selected for most of the testing were the instruments recording the lateral in-plane load versus displacement hysteresis, the corresponding sensors for lateral in-plane versus out-of-plane displacement, as well as the overall strains at the North and South ends of the wall specimens. In addition to the plots, a real-time value for each of the transducers and load cells was also displayed. This allowed for excellent monitoring of the wall specimen behaviour during the testing.

The signals from all channels were saved and scanned periodically. The data file was converted to individual channel files at every new cycle of testing to protect against data loss. After the testing was complete, all of the sets of channel files were compiled and downloaded on to a personal computer for processing and analysis.

3.5 Testing Procedure

The testing procedure was performed until failure of each wall specimen. Failure was defined for this testing as a drop in load of 50% of the maximum load resistance. Both wall specimens were of identical design and construction but were tested using different, although very similar, loading procedures. The test was manually displacement-controlled throughout the test, however for W1 a slaving matrix linked the loads in the vertical actuators, Actuators 2 and 3, to the load in the horizontal actuator, Actuator 1. The loading protocol was displacement-controlled and
consisted of one or more cycles of incrementally increasing lateral displacements in alternating directions simulating the behavioural of a real wall under lateral seismic loading. For W1, the target displacement for each cycle was expressed in terms of $\Delta y_1$, the yield deflection at the top of the wall specimen corresponding to the instance of first yielding of the outermost vertical reinforcing bars in W1 based on a moment-curvature analysis. The value of $\Delta y_1$ was also used as the loading increment for W2. This was done to keep the displacement increment levels identical despite the fact that the yield displacement of W2 was not identical to W1.

### 3.5.1 Loading Protocols

The loading protocols used during the testing were very similar between the two tests and were both altered real-time during the test to suit testing conditions. The protocols were based off of theoretical target displacement increments of $\Delta y_1$ at the top of the wall specimens and thus controlled by the displacement of Actuator 1. It is important to note here however that the displacements of Actuator 1 were actually occurring at 250 mm below the top of the wall specimens at the centre of the loading beam connection with Actuator 1. This was taken into account during the post-processing and analysis.

#### 3.5.1.1 W1 Loading Protocol

As was described earlier, for W1 the overall objective of the loading protocol was to simulate the effect of seismic loads on the lower portion of a 9 m high wall in a three-storey building (referred to as the simulated wall in this thesis). The simulated loads on the building were as if the building were a typical commercial structure for the first wall specimen, W1, to be tested. Each storey of the simulated wall in this structure was 3 m in height and supported 28 cm deep slabs, a live load of 4.8 kPa, a partition allowance of 1.0 kPa and a 0.5 kPa allowance for mechanical services, ceilings and finishes over a tributary length of 4 m. This resulting vertical compression loading of 660 kN (corresponding to axial compressive stress of $0.034 f'_m$) was held constant throughout the test to represent the axial gravity loads. The lateral loading from the seismic event had a simple triangular distribution at each floor level. Hand calculations of this load scenario yielded a $\Delta y_1$ value of 7 mm. Response 2000, Bentz and Collins (2000), a sectional analysis
program designed at the University of Toronto, was used to confirm the hand calculations. This value was indeed verified by the test on W1.

For the test on W1, before the main loading protocol had been initiated, an initial vertical compressive load representing the axial gravity loads on the wall specimen was applied by Actuator 2 and 3. This compressive load was applied in 10 increments of approximately 10% of the total load. The actuators were manually displacement-controlled in this initial stage of the test and each increment was applied to both actuators nearly simultaneously. Upon reaching 90% of the total permanent axial load, the actuators were set to force-control and the remaining load was applied. Applying the whole load at once would have caused a large load imbalance after the first vertical actuator load had been input and before the second actuator load could be input due to the manual nature of the process.

Once the vertical compression loads had been applied, the first lateral load increment was applied by inputting the increment into the displacement command of Actuator 1. The load induced in the load cell of Actuator 1 to create this displacement increment provided the load commands for Actuators 2 and 3 through the slaving matrix relationship described earlier. Depending on the direction of the displacement-increment, Actuator 2 or 3 would begin adding additional compressive load to the wall specimen and the other would be releasing an equal portion of its compressive load. The same process would occur as the wall specimen was loaded in the opposite direction.

The loading protocol was repeated for each target displacement increment cycle. Figure 3-28 illustrates the loading protocol utilized for the test on W1. The displacement level increments and the number of cycles at each increment were determined beforehand, however, changes were made during testing based on the performance of the wall specimen.

The displacement level increments were based on using target lateral displacement increment of \( \frac{1}{2} \Delta y_1 \) until a lateral displacement of \( 2 \Delta y_1 \) had been reached. At this point a target lateral displacement increment of \( \Delta y_1 \) was used until specimen failure. The number of cycles at each displacement level increment varied throughout the test. The increment of \( \frac{1}{2} \Delta y_1 \) was cycled only once. This single cycle was used instead of the conventional two cycles so as to not excessively degrade the wall specimen before large tensile strains were reached in the vertical reinforcement.
All increments afterwards were cycled twice, however, conditions of the specimen warranted the decision to return to a single displacement cycle after a compression failure was initiated at the wall toes at the $3\Delta y$ displacement level. This was done such that larger tensile strains could be reached in the vertical reinforcement before the strength degradation became too significant. The final target displacement was at $6\Delta y$ upon which the testing failure criteria was reached. Section 4.2 discusses this event in detail. The displacement-controlled Actuator 1 had a displacement rate which could also be controlled and modified during the testing. This displacement rate ranged from 0.5 mm/minute during the first cycle of the test and was steadily increased to a maximum of 3 mm/minute during the last cycle of the test. Actuator 1 would continue at this displacement rate until the target displacement had been reached. The displacement rates were increased as larger and larger target lateral displacement caused the cycles to take up much more time to complete.

![Figure 3-37](image)

Figure 3-37  Loading protocol developed for the testing on Specimen W1 ($\Delta y = \Delta_{y1}$). Note that the displacements in millimeters below the chart peaks describe the target maximum displacement for the corresponding cycle.
3.5.1.2 W2 Loading Protocol

The testing on W2 was designed to represent the worst-case scenario based on the observations made during the testing on W1 which suggested that high axial loads may help in the prevention of the out-of-plane instability failure mechanism. It was noted that the relatively small crack widths identified along the mortar joints during the testing were not sufficiently large enough to precipitate this desired failure mode. Therefore, the test on W2 was designed to not induce any axial load (except self-weight) onto the wall specimen. Although this may have been a worst-case scenario, this type of loading would still be considered typical of a single storey warehouse-type structure.

Due to simplification of the testing procedures with the absence of the vertical actuators the test was run much more easily. For the test on W2 the loading protocol was again based on the lateral displacement of Actuator 1 at its connection height to the loading beam. The loading increment was identical to that used for the test on W1 of 7 mm. Despite the fact that the lateral displacement corresponding to yielding of the first vertical reinforcing bar had theoretically increased for this second test, the previous value of \( \Delta y_1 \) was used as the displacement increment so that the behaviour of the two specimens could be closely compared overall. It is important to note however that the loading protocol of W2, shown in Figure 3-37, is labelled in terms of the adjusted, true \( \Delta y_2 \) of W2, that being the lateral load corresponding to the yielding of the first vertical reinforcing bar of W2 found during post-processing. Therefore, the drift ratios were equal for both tests at every increment number of the loading protocol sequence. This then effectively meant that drift ratio was acting as the parameter governing the loading protocols.

The loading protocol of W2 was also based upon that of W1 and the cycles were performed in the same manner and in the same directions. The displacement increment values as well as the number of cycles per increment were identical at the lower displacement level cycles of the W1 test. As the ductility capacity was much greater for W2 however, the displacement level cycles increased far beyond that reached in the W1 test as can be seen in Figure 3-30. At these higher displacement level cycles only one cycle was made at each increment. The displacement level increments increased at 0.2% drift until a total drift of 1.8% was attained. From this point until the end of the test, the drift increment changed to 0.4% drift due to damage occurring in the
specimen. Section 4.3 describes this transition in further detail. The displacement rates for W2 had a wider range than for the W1 test. These rates ranged from 0.5 mm/minute during the first cycle of the test and was steadily increased to a maximum of 10 mm/minute during the last cycle of the test.

Figure 3-38  Loading protocol developed for the testing on Specimen W2 (Δy = Δy2)

3.6  Conclusions

To achieve the objectives of this study, two RMSW specimens were constructed, both using the same 140 mm x 200 mm x 200 mm concrete masonry blocks and to the same overall height. The two specimens were designed and constructed identically with approximately the same material properties, detailing, and construction tolerances. The variance between the two tests was the
magnitude of the lateral loads and the presence of the applied axial compressive loading. This had numerous implications on the specimen behaviour as well their loading protocols. An analytical calculation of $\Delta_y$, the lateral displacement at the wall specimen top corresponding to the occurrence of yielding in the first vertical reinforcing bar, was used in the loading protocols as the primary displacement level increment. This theoretical value took into account the compressive and lateral resistance of the RMSWs. An analytical calculation of $\Delta_y$ was chosen rather than a measurement taken directly from the strain gauges due to these instrument's inconsistent reliability.

By increasing the target displacement level increments steadily higher it was possible to control the tensile strain in the vertical reinforcement which was a critical parameter to the development of an out-of-plane instability. Also, changing the applied axial load on the specimens demonstrated the extreme importance of this parameter as well. The loading protocols were very flexible which became very useful during the actual testing as events unfolded.
CHAPTER 4

Experimental Test Results

4.1 Introduction

In this experimental study, two identical RMSWs were constructed and tested using different loading/test setups to simulate the effect of seismic loads on the bottom storey of a tall RMSW. The specimens were subjected to combined lateral load, axial compression, and overturning moments. The first test (specimen W1) studied the effect of seismic loading on a 3-storey commercial building, whereas the second test (specimen W2) examined the effect of seismic loading on a single storey warehouse-type building.

A Summary of the Failure Mechanisms

One or more mechanisms of behavior were observed in specimens W1 and W2 during the testing. These mechanisms are described next.

1. Out-of-plane instability

Large tensile strains develop in vertical reinforcement across uniform, wide cracks extending along the plastic hinge length. The unsupported vertical reinforcement across these cracks may become unstable during a load reversal that leads to out-of-plane displacements. Instability of a RMSW occurs when out-of-plane displacements attain a critical value (half of the wall thickness). This mechanism occurred after high ductility had been reached in specimen W2, however the behaviour occurred at a ductility at significantly lower ductility level.
2. **Flexural cracking**

Flexural behaviour in these specimens was due to the high overturning moments relative to the applied shear forces and the aspect ratio of the wall specimens. Flexural cracking provided the first evidence of damage in both specimens, and it occurred at the ends of the specimens characterized by high curvatures and tensile strains in vertical reinforcement.

3. **Toe crushing**

Toe crushing occurred when the maximum load and ductility level was reached in both specimens. High curvature and corresponding axial strains and stresses developed in toe regions of the specimens. Crushing of the concrete block and grout would take place after the masonry compressive strength was exceeded. Toe crushing leads to a decrease in the compression zone and the flexural capacity of the specimens.

4. **Reinforcing bar buckling**

Face shell buckling of the vertical reinforcement occurred due to the lack of lateral support with the loss of masonry, and it usually accompanied the initiation of toe crushing and face shell spalling. Reinforcement buckling occurred in the toe regions subjected to compression. However, when buckled reinforcing bars were loaded in tension during the subsequent load reversal it would straighten nearly vertical position.

5. **Face shell spalling**

Face shell spalling is characterized by the loss of the concrete block face shell after they were loaded in excess of their capacity, and it takes place at the initiation of toe crushing. Face shell spalling was observed in both wall specimens. An apparent lack of bond between the grout and the concrete block units was also noticed as face shells spalled off and exposed grouted cores. Face shell spalling caused a drop in the vertical compression capacity of the specimens and consequently their flexural capacity. Face shell spalling continued to occur when higher ductility levels were reached in the specimen's face shell.

6. **Base sliding**

Base sliding occurred at significant lateral loading, and it was significant in the W2 specimen which was not subjected to applied axial compression. Unlike reinforced concrete shear walls, RMSWs are not characterized by a significant aggregate-interlock mechanism and the associated clamping forces in vertical reinforcement across the
horizontal crack at the wall base. As a result, RMSWs are more susceptible to base sliding mechanism. Base sliding resistance in RMSWs is provided by the Coulomb friction and dowel action in vertical reinforcement. During the test on specimen W2, the maximum sliding displacement along the base of the specimen was on the order of 30 mm. Since the specimens were designed for moderate axial compressive loading, base sliding was anticipated ahead of the test. Besides limiting the flexural capacity, sliding had other detrimental effects; for example, sliding displacement in the compression toe led to spalling of concrete in the footing, thereby causing a decrease in the wall capacity in the loading direction.

7. Concrete cover spalling in the footing
After significant base sliding was initiated in specimen W2, the wall toe had shifted over and past the location of the most exterior reinforcing loops in the concrete footing. When the fully intact wall toe began loading compression forces directly onto the cover concrete of the footing, the cover began to spall off due to the lack of reinforcement. This caused a non-symmetric response of the wall specimen in the North and South directions due to the reduced of the compression zone in this direction of loading.

8. Diagonal tension cracking
Stepped cracks along diagonal paths were observed in both specimens, especially in specimen W2 due to significantly higher ductility levels. The crack width was largest along the mortar joints, however a few cracks propagated through the concrete block units. Due to the significant amount of horizontal and vertical reinforcement in the wall specimens, the crack width remained relatively small (less than 0.5 mm). The cracking encompassed almost every mortar joint in specimen W2 and the majority of joints in specimen W1.

Experimental observations associated with the testing of wall specimens are presented in this chapter. Comparisons of the test behaviour as well as a focus on the out-of-plane displacement phenomena are discussed as well. Notes on the general behavior of wall specimens were taken throughout the tests. The crack widths were measured at the target displacement level for each cycle. This approach enabled me to observe the maximum crack widths during a particular cycle and smaller cracks which closed upon unloading in the subsequent cycle. The cracks were marked directly on each face of the specimen and the corresponding location was drawn on a
The crack diagrams for each wall face (east/west) and target displacement level were compiled to create a cumulative crack pattern for each specimen. The crack widths were measured using crack gauges.

### 4.2 Testing of Specimen W1 – Experimental Observations

A discussion on the behaviour of specimen W1 during the testing is presented in this section. Specimen W1 was loaded in such a way to simulate the effect of seismic loads on the lower portion of a 9 m high wall in a three-storey commercial building. The vertical compressive axial load was 660 kN (corresponding to an axial compressive stress of 0.034\(f'_m\)), and it remained constant throughout the test. A yield displacement \((\Delta_y)\) value of 7 mm \((\Delta_{y1})\) was calculated prior to the testing and the displacement increments were multiples of this value.

It should be noted that considerable discrepancies were observed between the target lateral displacements input into the command of Actuator 1 (actuator displacement feedback) and the actual response displacements recorded at the same location during the testing. It was observed that the recorded specimen displacement was perpetually lagging behind the target displacement. As can be seen in Figure 4-1, the differential displacement (difference between the target displacement and the recorded specimen displacement) was largest in the North direction, and somewhat smaller in the South direction. The largest discrepancies in displacement values were reported when the connection to the strong wall was in compression and smaller when the connection was in tension. This leads to the conclusion that, at least before the initial toe crushing failure took place, the differences were mostly due to the bending flat of deformed connection plates, possibly as a result of the aftermath of the heavy welds used. The other differences could be attributed to the connection gap closing within the test setup and the deflection at the top of the strong wall. Additionally, there would likely have been a fair degree of strain, and corresponding displacement, occurring in both Actuator 1's Dywidag connection rod to the strong wall and the connection rods transferring the loads from the loading beam to the wall. Base slippage of the reinforced concrete footing was negligible and did not contribute to the slippage.
Figure 4-1  Lateral in-plane displacement at the top of the specimen versus time (note a difference between the actuator displacement feedback/ target displacements and the recorded specimen displacements).

Figure 4-2 shows two main slopes of the lateral load versus differential displacement, that is, a difference between the target displacement and the actual specimen displacement, recorded in the initial stage of testing. This differential displacement can be observed in Figure 4-1 as the difference between the red and blue data series. The first slope may be due to the gap closing and plate bending, and the second slope may be due to the deflection of the strong wall. It should be noted that the largest differential displacement occurred after the peak load of 250 kN was reached. The instrument measuring slip between the loading beam and the wall specimen detected zero slip at every stage of the testing and thus this slippage may possibly be ruled out. These displacements may be due to additional rotation at the wall base as the toes underwent face shell spalling, and/or an increase in the loading rate a. The increase in loading rate was significant and may have caused a larger discrepancy between the displacement command and
displacement feedback from Actuator 1, thereby causing increased differential displacements during loading and unloading.

Figure 4-2  In-plane lateral load versus differential displacement at the initial phase of testing – specimen W1

The testing started when the vertical axial compression load of 330 kN was applied through Actuators 2 and 3. The load was applied in increments of approximately 10% of the total axial load in each actuator. Upon reaching 90% of the total load, the actuators were set to force-control mode and the remaining load was applied. During the application of the axial compression load, no cracking was observed in the wall specimen.

Lateral in-plane loading was applied through Actuator 1, according to the loading protocol shown in Figure 4-3. The first target lateral displacement level attained was the ½Δy increment in the North direction, applied at a 1 mm/minute rate. After the target lateral displacement was reached, the specimen was unloaded and the same target displacement was applied in the South direction. The specimen was then brought back to its initial position to conclude the cycle. This
loading protocol was followed throughout the test. A few minor step cracks formed in the lower half of the specimen, mostly along the mortar joints with the maximum width of less than 0.1 mm. The specimen remained essentially elastic at this displacement level.

During the following target lateral displacement level of $\Delta_y$ the specimen was subjected to two loading cycles. A number of cracks were observed, mostly minor stepped cracks originating at the ends of the specimen which propagated along the mortar joints in the lower half of the specimen, as shown in Figure 4-3. These cracks were small (maximum width of approximately 0.1 mm) and were widest along the mortar bed joints (horizontal joints). The behaviour of the wall specimen remained elastic at this stage of the testing.

At the $1.5\Delta_y$ displacement level, the cracks observed at the end of the previous displacement cycle ($\Delta_y$) further propagated, and a few new cracks were noted. The cracking pattern remained primarily in the form of stepped cracks that originated at the ends of the specimen in its lower
half. There were two instances of diagonal cracks extending through the block units (see Figure 4-3), however cracking mostly took place along the mortar joints. Maximum crack widths of 0.5 mm were observed at several locations.

![Cracking patterns at increasing target displacement levels for specimen W1](image)

**Figure 4-4  Cracking patterns at increasing target displacement levels for specimen W1**

During the $2\Delta y$ displacement cycle, additional cracks were observed along the end faces of the wall, and relatively few mortar joints remained uncracked. Horizontal cracks also began to open along bed joints at North and South ends of the wall, as well as along the base of the specimen. At this point, cracking was also observed in the footing but it was deemed not to affect the test results as the crack width remained below 0.3 mm throughout the test. Maximum horizontal and vertical crack widths of 0.8 mm and 0.1 mm, respectively, were measured along the existing cracks.
During the following cycle, the specimen was subjected to $3\Delta_y$ target displacement in the North direction. Extensive spalling developed at the North wall toe at the end of the cycle, as shown in Figure 4-4. The damage extended up the bottom two courses and resulted in a drop of 50 kN (20% of peak) of the lateral load. The specimen behaviour was similar during the next half-cycle when the $3\Delta_y$ target displacement was applied in the South direction. The South wall toe also experienced face shell spalling, and the load dropped by approximately 50 kN (20% of its respective peak). Cracks opened to a maximum 1 mm width along their horizontal lengths, and two new sets of step cracks were observed at the wall ends.

![Cracking at the target displacement $3\Delta_y$: a) North toe, and b) South toe](image1)

![Damage at the target displacement $4\Delta_y$: a) North toe, and b) South toe](image2)

Spalling at the wall toes became more extensive during the subsequent cycle with target displacement of $4\Delta_y$, as shown in Figure 4-5. At that stage, the wall compression zone shifted...
toward the centre because the toes crushed over the two bottom courses, and the vertical reinforcing bars at the wall ends were exposed. It was observed that the end reinforcing bars buckled under compression and straightened under tension. The plastic hinge region became apparent over approximately bottom four courses (800 mm height) starting at the wall base. Cracking along the bed joints at the North and South wall ends extended up the lower 11 courses. Several major horizontal cracks were observed on the West and East wall faces, with the maximum widths of up to 1.7 mm.

During the final target displacement load cycle of $6\Delta_y$, a significant loss of integrity of the wall toes was observed, as shown in Figure 4-6. Due to the compression failure at the North end, the specimen effectively lost the load-bearing capacity along 40% of its overall length (equivalent to 1000 mm). It was observed from the spalled face shells that the interior face shell surface in contact with the grout was clean and smooth, which is a sign of poor bond (this type of behaviour was also observed during the prism testing). During the same half-cycle, the buckled reinforcing bars at the South end nearly straightened under tension. Cracks along the horizontal bed joints opened to a maximum 2.5 mm width, while vertical crack widths remained within the 0.3 mm range. Besides additional large cracks at the wall toes, a new stepping crack formed at the compression end in the upper portion of the wall. The specimen demonstrated similar behaviour during the subsequent displacement half-cycle in the South direction. At this stage, crushing occurred in the South wall toe subjected to compression, and the reinforcing bars at the North end straightened under tension. The failure occurred near the end of this half-cycle, when a major drop in the load of 115 kN (roughly 60% of this cycle's peak load) was observed. After unloading to zero lateral load, crushing of the lower portion of the specimen took place along its entire length. It is estimated that an effective loss of the load-bearing capacity at the wall toes over five or more block lengths, combined with the presence of vertical loads, resulted in excessively high compressive stress (approximately $0.5f'_m$) at the centre of the wall. This length amounts to approximately 2.1 m, corresponding to 81% of the overall specimen length.
Figure 4-7  Failure of the specimen at the target displacement level of 6$\Delta y$: a) North toe, and b) South toe

Figure 4-7 and 4-8 show the load-displacement hysteresis curves, obtained by plotting the recorded lateral forces from the actuator load cell versus the in-plane lateral displacement near the top of the specimen (at the 18th course). The specimen displayed stable and relatively symmetrical hysteretic behaviour until the maximum load, $Q_u$, of 250 kN had been reached (corresponding to the displacement cycle 3$\Delta y$), as shown in Figure 4-7. The corresponding drift ratio was 0.36%, and the displacement ductility ratio ($\mu$) was 2.1. The equivalent yield load, $Q_y$, was determined by combining the hysteretic behaviour and the strain gauge data. The load-lateral displacement hysteresis provided a range of points where first yield may have occurred. After this range was established the strain gauge data allowed for a more refined determination of first yield. Figure 4-8 shows the final three load cycles preceding the failure. It can be seen that a 20% drop in load occurred at the 0.52% drift level, corresponding to the $\mu$ value of 3.1.

During the test, the specimen reached the maximum $\mu$ value of 4.6, and the corresponding drift ratio was 0.79%. Figure 4-9 shows the hysteresis curves for overturning moment versus lateral displacement (very similar to the curves shown in Figure 4-7). Figure 4-10 displays average strains in the exterior vertical reinforcing bars. Although the strain gauges detected vertical reinforcing steel strains on the order of 0.02, corresponding to 10 times the yield strain, vertical strains calculated from the recorded displacements indicated average strains of only 2$\Delta y$ over the entire wall height. The reason for this difference in vertical strains is that the vast majority of the strain was localized in the bottom two courses of the wall specimen. Also, it must be noted that
the strain was highly concentrated at the bed joints and thus the reinforcing steel experienced much more tensile strain than the surrounding masonry.

Figure 4-8   Lateral load-displacement hysteretic curve for specimen W1

Figure 4-9   Lateral load-displacement hysteretic curves for the last three loading cycles – specimen W1
Figure 4-10  Overturning moment versus lateral displacement for specimen W1

Figure 4-11  W1 hysteretic response of the specimen in terms of average vertical strains at the wall end zones
Key observations made during the testing of specimen W1 are summarized below:

1. The failure of specimen W1 occurred due to flexural compression (crushing) at the wall toes, and the out-of-plane displacement recorded during the test prior to the final failure was less than 3 mm.

2. An absence of the out-of-plane instability can be explained by relatively large axial compression stresses in the specimen caused by applied vertical loads. These stresses prevented the development of wide horizontal flexural cracks required for out-of-plane instability. The results suggest that the presence of high axial load might reduce the chances for lateral instability.

3. Buckling of laterally unsupported vertical reinforcing bars at wall ends was observed during the final loading cycles when the masonry disintegrated at the wall toes, however this did not precipitate global out-of-plane instability of the wall. These test results support the findings of Phase 1 study, that is, the presence of uniform, wide flexural cracks along the plastic hinge length is critical for the occurrence of global out-of-plane instability (Azimikor, 2012). It appears that localized wall toe crushing may in fact prevent out-of-plane instability from occurring.

4. High plastic tensile strains on the order of $10\Delta_y$ were recorded in the vertical reinforcing bars at the wall ends during the test, however average tensile strains over the wall height were significantly smaller ($2\Delta_y$). One of the findings of Phase 1 study was that high plastic tensile strain in the longitudinal reinforcement is the primary factor that determines the potential for out-of-plane instability in RMSWs.

- The maximum displacement ductility ratio ($\mu$) of 4.8 reached during the wall testing was comparable to the ductility levels achieved during the Phase 1 tests on uniaxial specimens which exhibited out-of-plane instability (3.2 to 6.3). The difference in failure mechanisms can be explained by the difference in strain gradients for the wall specimen and uniaxial specimens from Phase 1 study characterized by constant strain along the specimen length.

#### 4.3 Testing of Specimen W2 – Experimental Observations

Specimen W2 was loaded in such a way to simulate the effect of seismic loads on the lower portion of a single-storey warehouse-type structure. There were no applied vertical axial
compression loads representing gravity for this test. The same displacement increment value for the W1 test of 7 mm ($\Delta y_1$) was initially used in the loading protocol at the beginning of the test, however once the actual value of first yield of the most exterior reinforcing bar was determined to be 11.5 mm ($\Delta y_2$), multiples of this new displacement increment were utilized. The lateral in-plane loading was applied through Actuator 1, according to the loading protocol shown in Figure 4-12.

![Figure 4-12](image)

Figure 4-12 Loading protocol employed for tests on W2 showing lateral wall drift on the y-axis, target actuator displacements and the corresponding maximum crack widths for each photo. *crack width excluding toe crushing zones

A few measures were put in place to minimize the discrepancies between the lateral displacement command input to the horizontal actuator and the actual displacement at the top of the specimen recorded during the testing. The wall specimen was secured to the strong floor with
higher degree of post-tensioning, the support providing the base for lateral in-plane instruments were stiffened, and another instrument was added so that in-plane displacements could be recorded at both ends of the specimen. Despite these efforts, there was a negligible decrease in the discrepancy. This lead to a possible conclusion that the bulk of the discrepancy was due to simple gap closing, elongation of the Dywidag connection rods, and deflection of the strong wall. These factors may be impossible to eliminate but their effects should nonetheless be noted.

The behaviour was elastic during the initial displacement cycles ($0.3\Delta_y$, $0.6\Delta_y$, and $1.2\Delta_y$). Cracking occurred earlier than in specimen W1 test due to the absence of applied compression load, and cracks were observed starting from the first cycle ($0.3\Delta_y$ target displacement level). The cracks were mapped at the maximum displacement level during each loading cycle. Cracking was located primarily in the lower half of the specimen and it consisted mostly of horizontal cracks along the bed joints that originated at the bottom end of the specimen. Minor step cracking was also observed at the South end, originating at the edge of the fifth course of the specimen. The maximum crack width of 0.2 mm was observed along the bottom bed joint of the first course at the wall-footing interface. Of initial concern, a sliding shear crack, albeit small, formed along the length of the specimen at the third course. Figure 4-11 shows the crack patterns recorded at all displacement levels during the test on specimen W2.

After the $0.6\Delta_y$ displacement level, extensive cracking over the specimen height was observed, both in the form of horizontal cracks at the end faces and diagonal step cracks extending from the specimen end face to the middle portion of the wall. At a few locations, step cracks propagated through the corners of a few concrete blocks, however the vast majority of cracks were confined to the mortar joints. A long horizontal crack formed over the entire wall length, as the tension zone was quite long during each direction of the cycle loading. The sliding displacement caused by this crack was 0.6 mm. For the most part, the cracking was identical on both the East and West specimen faces, however a few additional horizontal flexural cracks along the bed joints formed during the second cycle (target displacement $\Delta_y$) bed joint The maximum crack width of 1.0 mm was recorded along several horizontal cracks. Vertical portions of the step cracks remained approximately 0.1 mm wide over the header joint heights.
At the $1.2\Delta_y$ displacement level, the cracking became even more extensive, with flexural and diagonal step cracks forming at almost every mortar joint in the lower 14 courses of the specimen. Another sliding shear crack formed at the base of the 14th course from the bottom. The crack originated as a flexural crack during the previous two cycles and now extended to encompass the entire specimen length. The second loading cycle at $1.2\Delta_y$ displacement did not cause significant additional cracking. The bottom two courses remained virtually uncracked throughout these two cycles. As with the test on specimen W1, cracks also formed in the footing during this cycle. The cracks were small (approximately 0.1 mm width) but were located at the North end of the footing and continued through both the East and West faces. Maximum horizontal and vertical cracks width were 1.3 mm and 0.25 mm respectively. The base sliding resulted in a maximum displacement of 1.2 mm.

At the $1.8\Delta_y$ displacement level, only a few new cracks formed but did precipitate the continuation of many diagonal step cracks along their path at the middle portion of the specimen. The crack widths continued to increase, and the maximum horizontal and vertical cracks of 2.0 mm and 0.3 mm were recorded. At this point the maximum sliding displacement at the base was 1.8 mm.

A very few mortar joints remained uncracked at the beginning of the $2.4\Delta_y$ displacement cycle, and a minimal additional cracking occurred during that cycle. However, vertical cracks were observed in the bottom two courses; these cracks extended from diagonal step cracks that originated in the upper courses. The existing cracks substantially widened, and the maximum horizontal and vertical crack widths were 4.1 mm and 0.5 mm respectively. There was also a relatively large increase in the sliding displacement at the base to the maximum value of 3.0 mm.
Figure 4-13  Crack patterns at increasing target displacement levels for specimen W2 (Notes: * = second cycle at displacement level; ** = crack widths excluding locations of face shell spalling)
During the $3.0\Delta_y$ displacement cycle, there was minimal additional cracking except for a few step cracks which extended through the corners of the concrete blocks. More cracks developed in the bottom two courses. The face shell spalling was initiated during this cycle. Cracks also began to form at the top of the South end of the footing, likely due to the sliding displacement at the base of the specimen of 5.6 mm achieved during this displacement cycle. This resulted in the end of the wall extending past the last reinforcing bar in the footing. Damage to the South end of the footing can be seen in Figure 4-13. The maximum horizontal and vertical crack widths had now reached 7.8 mm and 0.75 mm respectively. During the same displacement cycle, an increase in the out-of-plane displacements was noted. When the specimen began to be loaded in the South direction, out-of-plane displacements began to occur at very low levels of lateral load. This corresponds to low levels of axial load in the compression zone where the out-of-plane displacements were occurring, confirming the findings of Phase 1 of this research. As the lateral load increased (and hence the axial load in the out-of-plane displacement region), out-of-plane displacements due to unsupported vertical reinforcing bars were increasing until the cracks closed and the masonry began to stabilize the specimen laterally. At this point, the out-of-plane displacements would drop to zero. Figure 4-14 illustrates behaviour during this cycle.

![Figure 4-14](image1.png)  
![Figure 4-14](image2.png)

Figure 4-14  Cracking at the South toe and footing at the target displacement $3.0\Delta_y$: a) a view from the West, and b) a view from the East
The $3.6\Delta_y$ displacement level did not introduce new cracking in the specimen, except for the cracks that formed in the face shells of the North and South toes of the specimen. The cracks formed at South end of the footing in the previous cycle opened, and spalling of the concrete cover was observed. The sliding displacement at the base reached 14.3 mm and it likely precipitated the spalling. Horizontal crack at the wall base reached the maximum width of 10.5 mm, while the maximum vertical crack width was only 0.5 mm. Figure 4-16 shows the beginning of concrete cover spalling at the South end of the footing, as well as the large crack at the North toe of the specimen. The maximum out-of-plane displacement of 8 mm was recorded during this cycle in the West direction at the third course at the South end of the specimen.
At the $4.2\Delta_y$ displacement level, the first evidence of substantial out-of-plane displacements (greater than 10 mm) was observed. This cycle also generated the first face shell spalling at both toes, encompassing the same portion of the toe (a 100 mm long region between the vertical reinforcing bar and the end of the specimen). The face shell spalling was accompanied by the first instance of reinforcement buckling. The cracked portion of the South end of the footing completely spalled off, and the base sliding displacement remained unchanged (13.8 mm). The spalled portion of the specimen was removed, and it was not possible to measure total vertical strains at the South end of the specimen due to the loss of the instrument. The damage at the specimen toes can be seen in Figure 4-17. A maximum vertical and horizontal crack widths of 0.2 mm and 4.5 mm were recorded during this cycle. Out of plane displacements reached the maximum value of 17 mm.
During the $4.8\Delta_y$ displacement level cycle the amount of face shell spalling increased over the first course at the specimen toes. At this point the spalling became more significant and now spread to a distance 400 mm from the end of the wall specimen. The end reinforcing bar was completely unsupported over the height of the first course (approximately 200 mm). The bar buckled when subjected to compression, and straighten when subjected to tension. The support provided by the 180° hook of the horizontal reinforcement at the North end of the specimen did appear to help somewhat in maintaining stability of the vertical reinforcement. The cracking in the rest of the specimen remained virtually unchanged compared to the previous cycle. The maximum horizontal and vertical crack widths were 4.5 and 0.2 mm respectively. Base sliding displacement remained almost unchanged at 14.4 mm. Figure 4-18 depicts the damage of the specimen toes after this cycle. The out-of-plane displacement almost doubled to a maximum value of 30 mm, which was reported at the third course on the South end of the specimen.
During the $5.4\Delta_y$ displacement level cycle the spalling extended to the second course on the South end of the specimen. The spalling also spread over a 600 mm length at the South end and 400 mm length at the North end. Although considerable buckling of the exterior reinforcement was observed at both wall toes, buckling of the South end reinforcement was more significant, as can be seen in Figure 4-17. The North toe did not experience significant buckling, possibly due to the 180° hook at the end of the horizontal reinforcing bar at this end of the specimen (note that the other end of the same bar did not have a hook), as shown in Figure 4-19.

The out-of-plane displacements at the third course near the South end of the specimen reached the maximum value of 37 mm during the $5.4\Delta_y$ displacement level cycle. Sliding displacement at the base of the specimen in the North direction of loading increased to 16.5 mm at this stage of the testing. When the specimen was loaded in the South direction, the maximum sliding displacement was 14.1 mm; this might suggest a permanent displacement in the North direction.
Insignificant additional cracking was recorded as most of the deformations seemed to be concentrated in the toe. The maximum horizontal and vertical crack widths were 4.5 mm and 0.2 mm respectively.

During the $6.0\Delta_y$ displacement level cycle, additional spalling took place at both toes of the specimen. Buckling of the reinforcement continued when that respective toe was in compression and straightened when in tension. The damage is shown in Figure 4-20. The maximum sliding displacement was 18.6 mm. The maximum out-of-plane displacement reached 45 mm at the same location as in the previous cycle. Almost no additional cracking was observed and the maximum horizontal and vertical crack widths of 4.5 and 0.2 mm respectively (unchanged compared to the previous target displacement level).

![Image](image1.jpg)

**Figure 4-20**   Spalling and bar buckling at target displacement $6.0\Delta_y$: a) North toe, and b) South toe

Due to insignificant changes in the specimen performance during the previous displacement level, the displacement was increased to $7.2\Delta_y$. During this load cycle, damage at the North toe
increased significantly, with spalling extending up to the third course. This damage was attributed to out-of-plane displacement in the East direction of the masonry bonded to the exterior vertical reinforcing bar (roughly 200 mm in length), whereas most out-of-plane displacements on both ends of the specimen were in the West direction. Again, the presence of 180° hook at one end of the horizontal reinforcing bar may have prevented extensive damage at this location because the hook provided confinement for the end vertical reinforcing bar. The South end of the wall experienced a large out-of-plane displacement (60 mm) at the third course and near the end of the specimen. It should be noted that significant out-of-plane displacements (as high as 20 mm) occurred as high as the 12th course. Buckling and straightening of the bars became more apparent during this cycle and can be seen in Figure 4-21. The sliding at the base had now eclipsed the range of the sensor measuring these displacements with a 25 mm range. The amount of cracking in the specimen did not increase significantly, that is, the maximum crack widths remained the same as in the previous cycle.

Figure 4-21  Toe damage at the target displacement 7.2\(\Delta_y\): a) North toe, b) South toe bar straightening, and c) South toe bar buckling

The next (and final) displacement level, 8.4\(\Delta_y\), was intended to generate sufficiently high tensile strain in exterior vertical reinforcing bars to precipitate out-of-plane buckling. The displacement and the strain were calculated following the approach by Acevedo et. al (2010). During the loading in North direction, significant out-of-plane displacements occurred at the North end of
the specimen. A small (200 mm long) section of the wall bonded to the most exterior vertical reinforcing bar, displaced by over 100 mm out-of-plane in the East direction. The specimen moved in the West direction at lower magnitude, as shown in Figure 4-22.

![Figure 4-22](image)

**Figure 4-22** Damage to specimen at the target displacement $8.4\Delta_y$: a) overall damage; b) and c) out-of-plane displacements centred on exterior reinforcing bar – North toe

When the specimen was loaded in the South direction, out-of-plane displacements developed soon after the masonry at the South end of the specimen started to resist compression (a similar trend was observed in previous cycles). However, rather than the out-of-plane displacements eventually stabilizing and decreasing after the block face shells came into contact as was observed at lower displacement levels, they continued to increase beyond the theoretical critical out-of-plane displacement of 70 mm (half of the concrete block unit thickness). At this point, out-of-plane displacements became unstable and continued to increase with an increase in lateral in-plane displacements. A sudden drop in lateral load capacity prompted the conclusion of the test. A lateral displacement of only 62 mm was reached, compared to the 92 mm lateral displacement reached in the North direction. The damage was concentrated at the bottom three courses of the specimen, although significant out-of-plane rotations and displacements extended up to the 14th course. Out-of-plane rotations were highest within the bottom three courses, but remained substantial up to the eighth course. The out-of-plane instability failure was extensive.
and occurred over the entire specimen length, and was significant to a 1.6 m length (measured from the South end). Significant rotations were experienced around the middle of the specimen length. The straightening of the North end reinforcing bars caused the out-of-plane displacements at less than one-half of the displacements at the South end of the specimen. Figure 4-23 displays the aftermath of this failure mechanism, while Figure 4-24 displays the out-of-plane displacement versus lateral displacement during the final loading cycle. As can be observed, the out-of-plane displacements became unstable after reaching the critical displacement of half the wall thickness (70 mm in this case).
Figure 4-23 Out-of-plane instability at the target displacement $8.4\Delta y$: a) overall failure mechanism (on the previous page); b) failure along the specimen length; c) failure at the South toe; d) deformations of the end vertical reinforcing bars at the South toe, and e) out-of-plane instability failure viewed from the West.

Figure 4-24 Out-of-plane displacement versus lateral in-plane displacement at South end of specimen at the third course during the cycle with target displacement $8.4\Delta y$.

Load-displacement hysteresis curves obtained by plotting the recorded lateral forces from the actuator load cell versus the lateral in-plane displacement near the top of the specimen (at the 18th course) are shown in Figure 4-25 and 4-26. It can be seen from Figure 4-25 that the specimen displayed stable and relatively symmetrical hysteretic pinched behaviour throughout...
the test. However, a difference in the lateral load capacity of approximately 11% was observed in directions. The maximum lateral load in the South direction was lower - about 89% of the corresponding load in the North direction. This may be attributed to the crushing and spalling of the concrete footing in the South direction due to sliding at the base of the specimen. This action would have acted to reduce the capacity of the compression zone when the wall was loaded in this direction and therefore reduced its flexural capacity for this direction of loading relative to the opposite. The maximum load in the North direction, $Q_u$, was 223 kN, was reached at the displacement $3.0\Delta_y$, however it should be noted that the maximum load remained unchanged in the subsequent cycles (displacement $3.6\Delta_y$ and $4.2\Delta_y$). The maximum load of 199 kN was reached in the South direction at 0.64% drift. The corresponding drift ratios (0.86%) in both directions were identical; this corresponds to a displacement ductility ratio ($\mu$) of 2.67. Note that the equivalent yield load, $Q_y$, in both directions was determined from the hysteretic behaviour and supported by the strain gauge data. Figure 4-26 shows load-displacement hysteresis curves for the final seven load cycles before the failure. It can be seen that a 10% drop in the maximum load (relative to $Q_u$) in the North direction occurred at the 1.40% drift level, and the corresponding $\mu$ value was 4.36. This drop corresponds to the first instance of face shell spalling at the North toe of the specimen. The first large drop in the load capacity (7% of $Q_u$) in the South direction also occurred at 1.40% drift level. The maximum displacement ductility ratio ($\mu$) reached during the test was 7.85 and 6.62 in the North and South direction respectively. The corresponding drift ratios were 2.51 and 2.11% in the North and South direction respectively. Figure 4-27 shows hysteresis curves for overturning moment versus lateral displacement (very similar to the curve shown in Figure 4-25). Figure 4-28 displays average strains in exterior vertical reinforcing bars. Although strain gauges detected very high strains in reinforcing bars ($20\Delta_y$), average strains calculated from the recorded displacements were much smaller ($5\Delta_y$ or 10 mm/m). It should be noted that the average vertical strains were calculated over the wall height, although these strains were mostly concentrated at mortar bed joints. Recording of average vertical strains was discontinued when the instrument was removed after the concrete cover spalled in the footing at the South end of the specimen.
Figure 4-25  Lateral load-displacement hysteretic curves for the entire test – specimen W2

Figure 4-26  Lateral load-displacement hysteretic curves for the final seven loading cycles – specimen W2
Figure 4-27  Overturning moment versus lateral displacement hysteresis curves – specimen W2

Figure 4-28  Average vertical strains at the wall end zones – specimen W2
Key observations made during the testing on specimen W2 are summarized below:

1. Removal of axial compression load applied during specimen W1 testing enabled the development of wide flexural cracks over the specimen height; this was confirmed to be one of the critical factors for triggering out-of-plane instability in RMSWs.
2. Out-of-plane instability occurred after a significant ductility demand (on the order of 8) and drift ratio (approximately 2.5%) had been reached. The instability took place after the specimen experienced significant cracking and damage in the toe regions.
3. High plastic tensile strains on the order of $20\Delta_y$ were recorded in the vertical reinforcing bars at the wall ends during the test, however average strains over the wall height were significantly smaller ($5\Delta_y$); this confirmed the findings of Phase 1 of the study that high plastic tensile strain in the longitudinal reinforcement was a primary factor in determining the potential for out-of-plane instability in RMSWs.
4. Out-of-plane displacements were observed in several cycles preceding the failure. Once horizontal crack widths were large enough and distributed over several mortar bed joints, and after the exterior steel reinforcement experienced satisfactorily high plastic strains, out-of-plane displacements developed when compression loads were resisted by the exposed vertical reinforcing bars in the compression zone. Out-of-plane displacements were initiated at displacement of approximately $3.0\Delta_y$, corresponding to the displacement ductility ratio of 3.0. When the zone subjected to tension in the previous displacement cycle started to be loaded in compression, out-of-plane displacements began to occur at a very low lateral load level. As the tension zone transformed into the compression zone, the cracks closed and the out-of-plane displacements decreased, thus stabilizing the specimen. This behaviour was observed in all subsequent displacement cycles but with significantly increasing degrees as the ductility demands increased. Out-of-plane displacements would initially increase in each displacement cycle, largely as a result of widening of horizontal cracks and spalling at the specimen toes.
5. Instability of the specimen occurred when out-of-plane displacement reached the value of 106.5 mm; this exceeded the critical value of 70 mm (one-half of the wall thickness). This confirms the findings of Phase 1 testing and analytical models that instability occurs after out-of-plane displacements surpass the critical value.
4.4 Summary of Experimental Results

The results of the experimental study were important for improving the understanding of out-of-plane instability mechanism in slender RMSWs subjected to in-plane lateral loading. After examining the observations from the tests on specimens W1 and W2, it is possible to draw a few relevant conclusions regarding the design parameters which likely influence the occurrence of out-of-plane instability.

Specimen W1 was first loaded in axial compression, and it was also subjected to in-plane cyclic loading through increasing lateral displacements imposed at the top in combination with additional overturning bending moments. The loading eventually caused toe crushing, face shell spalling, and localized buckling of reinforcing bars at the wall toes when they were in compression. Many diagonal step cracks were observed during the testing. Horizontal flexural cracks at the wall ends were also observed, however they were characterized by smaller width compared to stepped cracks. Eventually, the specimen experienced damage at both toes and its lateral capacity was significantly reduced; this precipitated compression failure along its bottom courses.

Specimen W2 was laterally loaded at the top, however axial loading and supplemental overturning moments were not applied during the testing. Damage originated primarily in the form of horizontal flexural cracks and ultimately led to sliding at the wall base. Diagonal step cracks were also significant. Cracks were observed at almost every mortar joint by the time the specimen was subjected to higher displacement/ductility demands. Flexural crack widths were much greater compared to specimen W1, and may have precipitated the out-of-plane displacements that were initiated at relatively low ductility levels. In spite of damage in the form of toe crushing, face shell spalling, and localized buckling of longitudinal reinforcing bars, the specimen continued to behave in a ductile manner. Increasing out-of-plane displacements were observed in the compression zone of the plastic hinge region. The specimen encountered instability when out-of-plane displacements continued beyond the critical limit before stabilization could occur; this occurred after significant damage and ductility levels were reached.
A number of possible influencing factors can be established by observing the differences between the testing of these specimens. The primary difference was the type of applied loading used in the test; this significantly affected behaviour of the specimens. Both specimens (W1 and W2) were subjected to lateral in-plane loading at the top. However, specimen W1 was also subjected to axial compression and overturning moments. The following sections of this chapter will discuss the effect of differences in loading upon the specimen behaviour and in particular the occurrence of out-of-plane instability.

4.5 Comparison of Test Specimen Response

This section compares and discusses the results from the testing on both wall specimens described in the previous section. Particular attention has been paid to analyzing the out-of-plane instability failure mechanism and the potential factors influencing its occurrence. A comparison of the specimen behaviour includes a review of hysteretic curves for lateral load versus lateral displacement and overturning moment versus lateral displacement, the relative axial strains in the wall end regions, and the relationship of out-of-plane displacements at the end zones versus lateral displacements and axial strains.

4.5.1 Hysteretic Response

The behaviour of specimens W1 and W2 was significantly different during their respective tests. The primary difference between the testing procedures for the two specimens was the application of axial compression stress in specimen W1 which contributed to an increase in its lateral load capacity by approximately 14%, but ultimately led to more rapid masonry deterioration and a significant decrease in ductility by about 68% compared to W2. The flexural capacity in terms of overturning moment was also much higher in W1 than W2. The total flexural capacity, including the additional moment demand imposed by the vertical actuators, meant that the total overturning moment capacity was by 90% greater for specimen W1. Figure 4-29 shows superimposed load versus displacement hysteresis curves and Figure 4-30 shows superimposed overturning moment versus lateral displacement. A comparison of average vertical strains at the ends of the specimens is presented in Figure 4-31 (note that the loss of instruments attached to the South end face shells resulted in the lack negative of axial strain recordings in this direction). Figures 4-32 and 4-33 show hysteresis curves for out-of-plane displacements versus lateral displacements and axial strains.
axial strains. It can be observed from these figures that the out-of-plane displacement response was significantly different for specimens W1 and W2. The maximum out-of-plane displacement in W1 was only 3.1 mm, and it occurred prior to the compression failure at the South wall toe. The same value was eclipsed at the $3.0\Delta_y$ displacement level for specimen W2, which attained an out-of-plane displacement of 70 mm prior to instability and 106.5 mm prior to the failure.

![Overturning moment versus lateral displacement hysteresis curves for W1 and W2](image)

**Figure 4-29** Overturning moment versus lateral displacement hysteresis curves for W1 and W2
Figure 4-30  Lateral load-displacement hysteretic curves

Figure 4-31  Axial strains varying with in-plane lateral displacement
Figure 4.32  Comparison of out-of-plane displacement behaviour with regards to in-plane lateral displacement

Figure 4.33  Comparison of out-of-plane displacement behaviour with regards to axial strain
4.5.2 Failure Mechanisms

Although similar failure mechanisms were observed in specimens W1 and W2, the timing of their occurrence and the sequence in which they occurred was different. Specimen W1 performed in a less ductile manner than specimen W2.

During the first cycle of testing on specimen W1, diagonal tension cracks appeared along the bed joints and head joints along the middle half of the wall length and over the bottom half of the wall height. This was followed by additional step cracks extended to join flexural cracks formed at the edges of the wall specimen. The cracks propagated to encompass the lower half of the specimen height. Splitting and spalling of the face shells occurred at the wall toes; this was closely followed by the toe crushing, which forced the compression zone to move toward centre of the specimen length. Deterioration of the wall toes eventually led to the compression failure after which the specimen collapsed.

The cracking during the W2 test was initially in the form of horizontal flexural cracks at the wall edges and a horizontal sliding shear crack extending the entire wall length at the top of the third course. The subsequent cycles saw the development of a number of step shear cracks forming across the bottom two-thirds of the specimen height, as well as another long sliding shear crack forming along the bottom of the first course. During the following cycles, significant out-of-plane displacements were recorded. Cracking pattern remained the same; the existing cracks propagated and new cracks formed along all mortar joints in the bottom two-thirds of the specimen height. Face shell spalling was first observed at a wall toe, and it remained fairly insignificant until face shell spalling was observed at the other wall toe at a relatively high ductility demand (approximately $4.2\Delta_y$. As the ductility demand increased during the following target displacements, face shell spalling developed further and was accompanied by toe crushing. As observed during the W1 test, the crushing caused the compression zone to shift towards the centre of the specimen length; this prompted further face shell spalling and crushing in the compression zone. Eventually, out-of-plane displacements exceeded the critical limit of half the wall thickness (70 mm in this case), thereby prompting out-of-plane instability and failure.

By comparing the behaviour and failure mechanisms in the two specimens, it is evident that the compression loading incorporated in the W1 test caused the failure mode to tend towards a
combined flexure/shear (diagonal tension) failure, as opposed to the flexural failure observed in W2 (which was not subjected to compression stresses other than its self-weight). The presence of compressive stresses in W1 resulted in limiting crack widths and diminishing ability of the specimen to develop significant out-of-plane displacements. The lack of sizeable crack widths in W1 prevented substantial yielding in vertical reinforcing at the wall end zones and led to relatively brittle behaviour at the ultimate. However, the compressive stress did help to prevent the base shear sliding cracks in W1; these cracks were observed during the W2 test. Although similar losses in compression zone were observed in both tests, the presence of compressive stress in W1 led to compression/crushing failure in the specimen, after the effective wall area was effectively diminished due to toe crushing. However, a similar loss in compression zone caused a drop in lateral stiffness in the W2 end zones, which contributed to the out-of-plane instability which took place shortly thereafter out-of-plane instability.

Section 4.6.1 describes the behaviour of specimen W2 and the eventual instability failure Section 4.6.2 describes the differences between specimens W1 and W2, and design parameters which may have influenced the instability to occur in W2 but not in W1. Finally, general design criteria and restrictions for preventing out-of-plane instability in reinforced masonry shear walls are briefly discussed.

### 4.5.3 Specimen W2

As described in Section 4.5, out-of-plane instability failure mechanism was observed during the testing of specimen W2. Although out-of-plane displacements were observed during the W1 testing, the displacements were much smaller and not explicitly evident throughout the test. For that reason, the following discussion is focused on specimen W2.

Out-of-plane displacements in specimen W2 occurred at relatively low ductility level (corresponding to in-plane lateral displacements) and continued to increase until the failure, as shown in Figures 4-32 and 4-33. Crack widths, axial tension and compression of the wall end zone, as well as plastic strains in the vertical reinforcement were also positively correlated with the imposed lateral target displacements. This confirms one of the key findings of Phase 1 study - that these parameters have a significant effect on the occurrence of out-of-plane stability failure mechanism (Azimikor, 2012).
Distribution of out-of-plane displacements over the wall height at failure is shown in Figure 4-34. The out-of-plane displacement records during the W2 test are displayed in Figures 4-35 to Figure 4-41. Note that displacement profiles are shown at sections S1, S2, M, N1, and N2, as shown on a wall elevation in Figure 4-34. The same drawing shows seven recording locations (R1 to R7), which correspond to instrumented out-of-plane displacement points at the South end of the wall specimen (where the out-of-plane instability was initiated). The plots show that most significant displacements were recorded at location R3 at the centre of the third masonry course from the base. This is likely due to the combination of high tensile strains in vertical reinforcement at an adequate distance from the concrete footing which provided a near-rigid support to the wall. For a wall subjected to lateral load at its top, the maximum curvature develops at the bottom part of the wall (in this case, the bottom two courses), thereby causing significant tensile strains and cracking at the wall toes. Plastic tensile strains in vertical reinforcement combined with significant horizontal cracking at the wall toe are necessary requirements for developing out-of-plane displacements, as discussed in Section 2.6. Out-of-plane displacements were most significant in these regions (around wall toes).

Out-of-plane displacements at the lowest recording location R1 (first masonry course at 100 mm above the wall base) were relatively small. It can be seen from Figure 4-35 that the maximum displacements were on the order of 8 mm; these low values are likely due to the support provided by the vertical reinforcement embedded in the footing (a rigid boundary condition). This support appears to be significant, since it counteracted the effect of the large horizontal flexural cracks (widths greater than 10.5 mm prior to face shell spalling) at this location.

Out-of-plane displacements at the second recording location, R2, were more significant compared to R1 (see Figure 4-36). The displacements at this location followed a fairly regular pattern characterized by an increase in out-of-plane displacements until the maximum value was reached during a cycle with approximately zero lateral displacement. This corresponds to low axial load levels at the wall end zones as the curvature was close to zero; this is in line with the findings of Phase 1 study. Figure 4-42 illustrates this behaviour through diagrams showing relationship between axial load and axial strain reported in Phase 1 experimental study (Azimikor, 2012).
Recording location, R3 was the most important for the development of out-of-plane instability of the wall specimen. The out-of-plane displacements were the largest at R3 and also heavily influenced the distribution and value of displacements at the other recording locations. When the out-of-plane displacements exceeded the critical value (half the wall thickness or 70 mm) at R3, instability ensued followed by failure. It is interesting to note that the maximum and minimum residual out-of-plane displacements recorded at this location occurred in the opposite sequence of the other recording locations. During loading in the South direction, R3 would attain its maximum residual out-of-plane displacement whereas the other locations would record nearly zero displacement. When loading in the North direction, out-of-plane displacements would approach zero at R3, suggesting straightening of the South end of the wall. As this is opposite to the other recording locations, it seems that straightening of the wall specimen at recording location R3 led to an increase of displacements at other recording locations.

Very similar behavior was observed at recording locations R4 through R7. Hysteresis curves are nearly identical with only the scale diminishing as the height of the locations increased. The out-of-plane displacements at these locations were also significantly affected by the out-of-plane displacements and rotations at the bottom courses of the wall specimen. The out-of-plane displacement was held nearly constant at the top of the specimen by the out-of-plane supports. There was very little transverse rotation above the plastic hinge height of the wall. This explains an almost linear decrease of displacements at locations R4 to R7.
Figure 4-34   Post-failure buckled wall shape at different sections (cross-sections are from South end perspective)
Figure 4-35  Out-of-plane displacements at recording location R1 (see Figure 4-34)

Figure 4-36  Out-of-plane displacements at recording location R2
Figure 4-37  Out-of-plane displacements at recording location R3

Figure 4-38  Out-of-plane displacements at recording location R4
Figure 4-39 Out-of-plane displacements at recording location R5

Figure 4-40 Out-of-plane displacements at recording location R6
Figure 4-41  Out-of-plane displacements at recording location R7

Figure 4-42  Applied axial load vs. axial strain over the plastic hinge region (left) and applied axial load vs. maximum lateral deformation at midheight normalized with respect to thickness (Azimikor 2012)
4.5.4 Implications on Possible Restrictions to Prevent Out-of-Plane Instability

This section looks briefly into the likely effects of the applied axial stress on the design parameters, as well as possible restrictive measures for preventing out-of-plane instability.

Due to identical design of the test specimens and lateral loading test setup and protocol, the applied axial stress induced on W1 during testing is apparently the governing factor causing different behavior in these two specimens. The applied axial stress affected the wall specimen behaviour significantly and in several different facets. The primary effect was a decrease in vertical tensile strains across the wall length during lateral loading. This increased both the lateral load capacity as well as the bending capacity of W1 compared to W2. The sliding displacements observed in the W2 test were also prevented during W1 testing due to the applied axial stress. In regards to out-of-plane stability, the decreased tensile strains acted to reduce flexural crack widths as well as the degree of plastic tensile strains in the vertical reinforcement. As described in Section 2.6, a decrease in vertical strains at the wall end zones minimizes the chances of out-of-plane instability.

As described by Azimikor (2012), inelastic tensile strain demand, $\varepsilon_{id}$, is the most practical parameter for use in design applications. This strain demand can then be compared with the estimated maximum tensile strains leading to out-of-plane instability. As a first step in determining the inelastic strain demands, specific curvature ductility factors for given ductility demands must be obtained.

In order to obtain the curvature ductility for a slender shear wall, a total displacement demand needs to be determined as a sum of elastic and inelastic lateral displacements, $\Delta_p$ and $\Delta_y$ respectively, as shown in Figures 4-43 and 4-44. Inelastic tensile strain demand, $\varepsilon_{id}$, can be determined from Equation 2-8 assuming linear curvature distribution across the wall length and linear plastic deflections above the plastic hinge region (Azimikor, 2012)

$$\varepsilon_{id} = 0.85 \left[ \frac{2h^2}{3l_p(2h - l_p)} (\mu - 1) + 1 \right] (\varepsilon_{sy} + \varepsilon_{my})$$ 2-8
Figure 4-43  Curvature distribution in a RMSW section: a) wall cross-section; b) yield curvature, and c) ultimate curvature (adapted from Anderson and Brzev, 2009)

Figure 4-44  Shear wall at the ultimate strain: a) wall elevation; b) bending moment diagram; c) curvature distribution; and d) elastic and plastic deflections (Anderson and Brzev, 2009)
A presence of applied axial stress will lead to a decrease in the total tensile strains, as shown in Figure 4-45. Note that the tensile strain limit, \( \varepsilon_{id} \), does not change, however for a given overturning bending moment on a given wall, the total tensile strains will be reduced and the wall specimen will have an increased overturning moment capacity.

The compression loading also causes an increase in the masonry axial stress and forces it beyond its ultimate strain before enough tension can be developed on the opposite of the end of the wall to result in vertical reinforcement yielding. The out-of-plane failure mechanism is preceded by a ductile flexure-controlled behaviour; this was reported by other researchers, as discussed in Chapter 2, and was confirmed during specimen W2 testing.

A presence of applied axial compression stresses also causes an increase in the length of the wall compression zone; this is believed to have effect on the occurrence of out-of-plane instability. This can be explained by the fact that the remaining portion of the wall provides lateral support to the compression zone. As the compression zone lengthens, a larger portion of the wall length is subjected to the out-of-plane instability, as described in Section 2.6. Over the wall cross-section, largest out-of-plane displacements would likely occur near the edge of the wall and approach zero towards the tension toe. If the wall was at some point subjected to compression

\[
\varepsilon < \varepsilon_{id} \quad \rightarrow \quad \varepsilon > \varepsilon_{mu}
\]
over its entire length, the wall would be most vulnerable to possible out-of-plane instability; this would be similar to compression loading applied to specimens tested in Phase 1 study. Conversely, if the wall specimen was entirely in tension, this failure mechanism would obviously not occur.

This poses an interesting dilemma. As the compression zone lengthens, the tension zone shortens; therefore, large tensile strains occur over a smaller length. Although a longer portion of the wall is subjected to compression during the subsequent load reversal, the only portion of the wall susceptible to out-of-plane displacements is still confined to the length previously experiencing high tensile strains. Hence, an increase in the compression stress on the wall will actually decrease a probability of out-of-plane instability.

Further research is needed to determine viable limitations on wall design parameters that would prevent occurrence of out-of-plane instability, however a number of relevant parameters have been identified for future consideration. The most likely is a strain limit placed on the extreme tension fiber of a RMSW as described by Azimikor (2012). Concrete and masonry design standards already impose related limits on curvature ductility, it is thought that this approach might be feasible for out-of-plane instability. Alternatively, it would be possible to restrict the applied axial stress level due to the effect it has on the compression zone length and failure mode type of a RMSW under seismic loading, as reported in this study.
CHAPTER 5

Conclusions and Future Work

5.1 Summary

The reversed cyclic behaviour of reinforced masonry shear walls (RMSWs) was investigated experimentally in this study. Out-of-plane instability was explored in regards to its existence in slender RMSWs and the height-to-thickness ratio restrictions imposed by the Canadian masonry design standard, CSA S304.1-04. The experimental results demonstrated significant influence of applied axial stress on the formation of wide uniform cracking over the specimen plastic hinge height and the associated tensile strains in the vertical reinforcement. These factors were found to be of significant consequence on the occurrence of out-of-plane instability. The results of this study were also used to verify findings of Phase 1 of this research program in which reinforced masonry uniaxial specimens, intended to represent the end zone of a RMSW subjected to gravity loads and in-plane lateral loads, were subjected to reversed cyclic axial tension and compression.

This experimental study included the construction and reversed cyclic testing to failure of two full-scale RMSWs with an aspect ratio of approximately 1.5 and a height-to-thickness ratio of 27, far exceeding the limits prescribed by CSA S304.1-04. Specimen W1 represented the bottom storey of a three- to four-storey building while the specimen W2 represented a tall wall in a single storey warehouse-type structure. The primary difference in the test setup for these two specimens was the application of an applied axial stress in specimen W1. Both specimens were...
tested using the same loading protocol and displacement increments until the specimens had lost half of their peak lateral load capacity.

The experimental testing took place at the Structures Laboratory of the University of British Columbia while materials testing was conducted at the Materials Laboratory of the Department of Civil Engineering, British Columbia Institute of Technology. The conceptual and detailed design as well as construction of the wall specimens, design and construction of the test setup, layout and installation of instrumentation, and data processing were performed by the author of this thesis.

The findings of experimental testing performed in this study contributed to understanding of the factors that influence the out-of-plane instability phenomena in RMSWs, and contributed to limited body of knowledge related to out-of-plane instability failure of reinforced masonry and reinforced concrete shear walls subjected to in-plane reversed cyclic loading. The experimental study has provided essential benchmark test data which will be used for critical calibration, validation and improvement of analytical models for out-of-plane instability in RMSWs planned for the next phase of this research program.

5.2 Key Findings Related to Out-of-Plane Instability of RMSWs

Both specimens had wall height-to-thickness ratios well above the CSA S304.1-04 limits, indicating that they may be susceptible to out-of-plane instability. However, out-of-plane instability occurred only in specimen W2 which was not subjected to applied axial stress and at ductility demands far higher than expected from RMSWs according to CSA S304.1-04 and the National Building Code of Canada 2010. Key findings of the experimental study are summarized below.

1) The effect of applied axial stress

During the testing it was readily apparent the failure mechanisms leading up to the final failure mode varied depending on the applied axial stress. The out-of-plane failure mechanism was
initiated after a ductile, flexure-controlled mechanism developed. The specimen experienced significant damage before out-of-plane displacements became critical.

The applied axial stress also acted to increase the length of the compression zone and in turn decrease the length of the tension zone causing smaller portion of the wall length being susceptible to out-of-plane instability. This concept of wall length susceptible to out-of-plane instability is based on the rest of the wall length supporting the portion undergoing out-of-plane displacements.

Although toe crushing is a ductile failure mode and was observed in both wall tests, degradation of the toe integrity was much more rapid when axial compression was present. This was in the form of face shell splitting and spalling as well as loss of the compression zone material. The compression zone would shift from the edge of the wall specimens towards the centre and act to reduce the lateral load capacity. The exposed vertical reinforcement was then vulnerable to local buckling which is thought to influence the out-of-plane displacements and eventual instability. While out-of-plane displacements did initiate at low levels of ductility when the toe regions were still intact, the largest out-of-plane displacements occurred after the vertical reinforcement was essentially unsupported over the bottom two course heights (approximately 400 mm) and across a length of almost 600 mm. The unsupported lengths may have produced larger out-of-plane displacements of the wall end zone than if support was provided by intact concrete block units and grout, thus prolonging the attainment of the critical out-of-plane displacement causing instability.

2) Uniformly distributed wide horizontal cracks combined with large plastic tensile strains in the vertical reinforcement required to trigger out-of-plane instability in RMSWs

Crack width and a uniform distribution proved to be important factors required to trigger the onset of out-of-plane failure mechanism. This finding is primarily related to horizontal cracks along mortar bed joints that are caused by flexural failure mechanism. As the crack width increased, plastic tensile strains in vertical reinforcing bars crossing these cracks also increased. Upon load reversal, the crack widths would reduce in size but would remain open due to the presence of the plastic strains. At this stage, lateral stiffness of the wall end zone relied only on exposed vertical reinforcing bars crossing the cracks; note that these bars also had to transfer the
compression stress across the crack to the grouted masonry cores. This caused transverse rotations across these cracks and out-of-plane displacements in the lower portion of the wall at the end zones.

It was noted that even the high tensile strains in the vertical reinforcement recorded during the W1 test (over $10\Delta_y$ at some mortar joint locations), were not enough to precipitate out-of-plane instability. Only when the strains reached levels as high as $20\Delta_y$ in the W2 test did instability occur. It is important to note that these strains are from local recordings at the mortar joints by strain gauges and that the vertical strains were significantly lower when averaged over the entire wall height ($2\Delta_y$ and $5\Delta_y$ for W1 and W2 respectively).

3) Critical magnitude of out-of-plane displacements

If large crack widths occur at uniform locations along the mortar bed joints, the lateral stiffness of the wall end zone would be similar to an unsupported reinforcing bar spanning over the plastic hinge height. The ensuing out-of-plane displacements would increase until the crack closure takes place. There are two possible scenarios depending on the timing of crack closure: a) if out-of-plane displacements increased beyond a critical limit (found to be approximately equal to half the wall thickness), they would increase in an unstable manner leading to instability of the wall specimen, and b) if the cracks manage to close before the displacement reach the critical limit, masonry compressive stresses would develop at the edge of the crack, thereby providing a restoring moment to the wall thickness. This would eventually lead to decreasing out-of-plane displacements and stability would be restored. This finding is in agreement with the Phase 1 study results discussed in Section 2.6. The mechanism of out-of-plane instability is illustrated in Figure 2-4.

During the test on W2, out-of-plane displacements were observed in many cycles preceding failure. This was because the cracks had not opened enough to permit this critical out-of-plane displacement. Evident out-of-plane displacements first initiated at a low ductility demand of $3\Delta_y$ when compared to the final instability failure at $8\Delta_y$, seven displacement cycles later.
4) Significant displacement ductility demands required to trigger out-of-plane instability

Large in-plane lateral displacements are needed to achieve these necessary crack widths and distribution in order to cause out-of-plane displacements great enough to precipitate instability. For a wall specimen without applied axial loading (W2), these lateral displacements could only be achieved at high ductility demands (corresponding to lateral displacements on the order of $8\Delta_y$). In case of specimen W1 subjected to relatively low level of axial compression (approximately $0.03f_m$), crack widths and tensile strains in vertical reinforcement were prevented from reaching required magnitude before another failure mechanism caused collapse of the specimen.

5) Wall height-to-thickness ratio not the only parameter for out-of-plane instability

From the results of the current experimental testing it is apparent that the height-to-thickness ratio alone may not be the optimal governing criterion for the occurrence of out-of-plane instability. Future experimental testing and analysis are required to better understand factors and parameters that may be more suitable for evaluating chances for out-of-plane instability in RMSWs.

5.3 Future Phases of Study

As noted throughout this thesis, the experimental research presented here is a part of larger experimental study. The experimental results presented here, along with the experimental and analytical results presented in the first phase of the study by Azimikor (2012), will be used for the design and analysis of future experimental testing and analytical modelling conducted at the University of British Columbia. The next phases of the study will be focused on further determining factors and parameters affecting the out-of-plane instability failure mechanism. After the numerous tests on a number of RMSW specimens of different designs and applied loading, the experimental test data will be used in the development of an analytical model capable of capturing the out-of-plane failure mechanism as it occurs in RMSWs. The results of the next phase of the investigation is expected in 2015.
5.3.1 Future Experimental Testing

The testing conducted in this current phase of the research study provided an essential benchmark for future experimental testing used to investigate the out-of-plane instability failure mechanism and how it occurs in RMSWs. As described earlier in this chapter, the main purpose of this experimental study was to better understand the mechanism and to observe the effects of axial loading on the tensile strains as well as the crack widths and distribution. The next phase of this research study will further investigate influencing factors on the out-of-plane instability failure mechanism. The future testing will provide additional experimental data to not only confirm the observations made during this current study, but also to provide recommendations for future masonry design standards.

The following recommendations for future experimental testing should be considered:

- Further investigation should be invested regarding the influence of height-to-thickness ratio on the occurrence of out-of-plane instability. This may be done by performing a number of additional tests on RMSWs of varying height-to-thickness ratios to observe whether or not out-of-plane instability occurs at lower ductility levels for wall specimens with higher height-to-thickness ratios.

- The effect of the amount and layout of both vertical and horizontal reinforcement should be explored. Variation of reinforcement ratios as well as reinforcement spacing should be done for a number of wall specimens to examine how this may affect out-of-plane instability. It is important that "industry typical" reinforcement be incorporated in the testing as these will be the most common types of walls used in practice. This would affect reinforcing bar diameters and spacing as well as possible dowel connections to the concrete footing of the wall specimens.

- Aspect ratio is another parameter to be investigated. It is thought that more slender walls (characterized by higher aspect ratios) may be more susceptible to out-of-plane instability. Therefore wall specimens with both lower and higher aspect ratios should be tested to observe differences in the out-of-plane displacement behaviour.
• Wall layout and configuration is also thought to have a significant effect on out-of-plane instability. Incorporating short flanges or boundary elements to a wall of rectangular cross-section provides stability to the wall end zones and offers a possible solution for out-of-plane instability of RMSWs. On the other hand, walls with T-shaped cross-sections may be more susceptible to out-of-plane instability. This is because very high tensile and compressive strains may develop in the wall web, which are believed to increase susceptibility to out-of-plane instability.

• Axial loading was found to be very influential to out-of-plane instability in a number of different ways. Vertical strains are believed to be the most important parameter affected by axial loading. Therefore, varying degrees of axial loading should be applied to further investigate its influence and possibly obtain a critical axial stress which may help to prevent out-of-plane instability.

• The number of layers of vertical and horizontal reinforcement may also affect the occurrence of out-of-plane instability. Increasing the number of layers (likely not more than two) may help prevent large out-of-plane displacements leading to instability or delay it until higher ductility levels are achieved.

• How the horizontal shear reinforcement is anchored and the location of the bond beam courses in which they are located may influence out-of-plane instability. An observation made during the current testing suggested that horizontal reinforcing bars with 180° hooks around vertical reinforcement appeared to help stabilize the vertical reinforcing bars. This is unconfirmed but worthy of note for further experimental testing. As horizontal reinforcement is contained within grouted bond beam courses, it may be important to note the locations of these bond beams. In current standard practice, bond beams are not located at the bottom course of a wall, but this may help in decreasing its susceptibility to out-of-plane instability.

• Although this is likely not the case in practice, reinforcing bar place holders should be used to ensure the reinforcement remains at the centre of the grouted cores during construction. This would allow the investigator to ignore the out-of-straightness effects on out-of-plane instability. Similarly, vertical bar placement could be varied for different
wall specimens to look more closely at how out-of-plane instability is thus affected. This may be a worthwhile investment as Phase 1 of this research study noted the importance of this parameter.

- Furthermore, it is recommended that instrumentation modifications are made such that out-of-plane displacement and transverse rotation profiles may be recorded over the height of the wall specimens. This would not only allow for more accurate results by neglecting local face shell spalling and other damage affecting the instruments, but also enable easier data processing. The same system may also be used for the overall lateral displacement behaviour of the wall specimens as well. One possible system may be Particle Image Velocimetry (PIV). The principle of PIV is simple and allows the tracking of a number points on the wall specimen over the course of the testing and would be highly useful for the proposed applications.

5.3.2 Future Analytical Models

The goal of future analytical models should be to develop a rational analysis tool that is able to describe the out-of-plane instability failure mechanism as it occurs in RMSWs subjected to lateral loading. Upon the development of such a model, it must be validated with experimental data in order to establish acceptable height-to-thickness ratio limits or restrictions on other parameters governing out-of-plane stability. Development of this model requires a sophisticated model taking into account many phenomena acting together. The following are recommendations for this future analytical model:

- The selected model must have the capacity to simulate the inelastic response of RMSWs as well as the out-of-plane instability failure mechanism. One possible macroscopic analytical model could be based on the MVLEM element (Multiple Vertical Line Element Model) developed by others. This model will require modification to capture the out-of-plane displacement behaviour by changing the uniaxial springs in the wall end zones to include second order effects to simulate instability in the wall compression zone. The properties of the uniaxial elements must be calibrated from the findings of Phase 1 of this study on the uniaxial cyclic response of reinforced masonry columns. This may be done by first formulating a fiber element which accurately reproduces the observed
responses of Phase 1 and then incorporating this element directly into the MVLEM element as one of the uniaxial spring elements. Computational platforms such as OpenSees (2009) would provide such modelling capabilities.

- A number of phenomena regarding the out-of-plane failure mechanism must be modelled in order to accurately capture it. This includes how the uniaxial elements interact with one another to represent the way in which the portion of a wall undergoing out-of-plane displacements is supported by the remaining wall length. This may require the use of additional lateral spring elements to provide connectivity. A mechanics-based analysis procedure for establishing the criteria for out-of-plane instability in RMSWs must be developed in order to compare the buckling capacity of the wall compression zone and the resultant compression force. This must account for the degradation in stiffness due to previous tension loading as well. Appropriate geometric transformation types are important for this modelling and a variety of types may be investigated.

- Another alternative to using fiber elements for the uniaxial spring elements would be to use a combination of rigid link, beam-column, and gap elements to act together in describing out-of-plane instability behaviour of a uniaxial reinforced masonry element. This method would take advantage of the tendency for the deformations in masonry to be located primarily at the horizontal mortar joints while the concrete block elements remain elastic. This method may be susceptible to convergence issues due to the number of rigid elements involved but it may also be more versatile option. The definition of the gap element properties and their behaviour would be critical to the accurate representation of the out-of-plane instability. The use of initial crack widths may be an option in order to neglect the mortar joint thicknesses and simplify the model.

- Irrelevant of the analytical model used, a number of important parameters will likely have a strong effect on the models ability to represent out-of-plane instability behaviour. Boundary conditions, which were not completely certain during the Phase 1 testing, are very important to the development of out-of-plane displacements. How these conditions are represented can drastically change the displacement distribution as well as the maximum values. The location of the reinforcement within the concrete block unit cells is
critical as the eccentricity of the vertical reinforcement may govern the stability of a reinforced masonry element entering compression after undergoing tension. As the Phase 1 testing had varying degrees of eccentricity between and within specimens, this parameter may require significant attention. Reinforcement stiffness at the crack is another issue of concern. Depending on the effective length of the reinforcement around the assumed mortar joint crack, significant changes to the vertical strains may result. These strains of course have a large effect on the out-of-plane displacement phenomena. Also of importance is the ability of the model to represent damage accumulation. It was found that damage accumulation, especially at the wall toes, may have a discernible effect on out-of-plane instability and must therefore be taken into account.
Bibliography


American Society of Civil Engineers (2006). Seismic Rehabilitation of Existing Buildings, ASCE-41, ASCE World Headquarters, Reston, Virginia, USA.


TMS 402-08/ACI 530-08/ASCE 5-08. (2008). Building Code Requirements and Specification for Masonry Structures, American Concrete Institute, ACI, Farmington Hills, Michigan, USA.


