THE OUT-OF-PLANE BEHAVIOUR OF REINFORCED CONCRETE
MASONRY WALLS WITH WINDOW OPENINGS

A research report
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Abstract

The aim of this research was to construct and test a partially grouted reinforced concrete masonry wall with window openings under out-of-plane loading and evaluate its behaviour. The behaviour of the wall was compared with the results of various theoretical models, and the accuracy of these models was evaluated relative to the experimental results.

The specimen was 9.4m long with 1.8m long return walls at each end, giving it a C-shaped plan. Windows were sited to reduce the out-of-plane strength – a corner window at one end and a window that sat astride the 45° yield line near the other end. 20-series concrete blocks were used to form the wall, with D12 vertical reinforcing at 800mm centres, giving a vertical steel ratio of 0.074%, and 2 D16 bars horizontally forming a bond beam in the two upper masonry courses. The wall was partially grouted, with only cells containing reinforcement filled. The steel used was the minimum required by NZS 4229: 1999, with some minor modifications. The wall was founded on a Ribraft waffle slab flooring system.

The test specimen behaved in a ductile manner, with a highly pinched hysteresis response. The ultimate strength of the wall was 5.8kPa at 2% interstorey drift, which is specified in NZS 4203: 1992 as the maximum allowable structural displacement for this type of structure. This is 40% higher than the maximum calculated ultimate limit state loading of 3.5kPa. Load carrying capacity did not decrease at very high post-elastic deflections. Because of this, it has been determined that the design of partially grouted masonry walls for out-of-plane loading is governed by deflection, not strength, requirements.

Different analysis techniques were used to calculate the response of the wall prior to testing. Finite element modelling in ABAQUS provided a good match with experimental data. Yield line theory was found to give a conservative value for the ultimate load. A modification to yield line theory to model load-displacement response was proposed, with results obtained from a simple model giving reasonably accurate results at high deflections.
Acknowledgments

This research project was constructed in the Civil Engineering Department of the University of Canterbury. I would like to thank the Department for providing the laboratory space, technical assistance and funding that made this project possible.

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<td>$\alpha$</td>
<td>maximum ratio of net block area to gross masonry unit area</td>
</tr>
<tr>
<td>a</td>
<td>depth of compressive block</td>
</tr>
<tr>
<td>$A_e$</td>
<td>½ the area of the smallest box that can enclose a hysteretic loop</td>
</tr>
<tr>
<td>$A_h$</td>
<td>area enclosed by a hysteretic loop</td>
</tr>
<tr>
<td>$A_i$</td>
<td>area of the $i$th panel</td>
</tr>
<tr>
<td>$A_s$</td>
<td>area of steel</td>
</tr>
<tr>
<td>b</td>
<td>width of masonry strips</td>
</tr>
<tr>
<td>$C_{pi}$</td>
<td>basic horizontal coefficient for a part at level $i$</td>
</tr>
<tr>
<td>d</td>
<td>depth to steel</td>
</tr>
<tr>
<td>$d_b$</td>
<td>reinforcing bar diameter</td>
</tr>
<tr>
<td>$\Delta_i$</td>
<td>centrodial displacement of the $i$th panel</td>
</tr>
<tr>
<td>E</td>
<td>reinforcing steel modulus of elasticity</td>
</tr>
<tr>
<td>$E_m$</td>
<td>masonry modulus of elasticity</td>
</tr>
<tr>
<td>$\varepsilon_{sh}$</td>
<td>reinforcing steel strain hardening strain</td>
</tr>
<tr>
<td>$\varepsilon_{u}$</td>
<td>reinforcing steel ultimate strain</td>
</tr>
<tr>
<td>$\varepsilon_y$</td>
<td>reinforcing steel yield strain</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>concrete cylinder compressive strength</td>
</tr>
<tr>
<td>$f'_{cb}$</td>
<td>masonry block compressive strength</td>
</tr>
<tr>
<td>$f'_{cr}$</td>
<td>modulus of rupture for masonry strips</td>
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<td>$f'_g$</td>
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<tr>
<td>$\varphi_{cr}$</td>
<td>cracking curvature</td>
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<td>$F_{ph}$</td>
<td>horizontal seismic pressure load</td>
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<td>ultimate curvature</td>
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<td>$\varphi_y$</td>
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<tr>
<td>$I_{cr}$</td>
<td>cracked moment of inertia</td>
</tr>
<tr>
<td>$I_e$</td>
<td>effective moment of inertia at $M_a$</td>
</tr>
<tr>
<td>$I_g$</td>
<td>gross moment of inertia</td>
</tr>
<tr>
<td>L</td>
<td>unknown length in yield line analysis</td>
</tr>
<tr>
<td>$L_w$</td>
<td>width of the window</td>
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<tr>
<td>Symbol</td>
<td>Description</td>
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<tr>
<td>$L_x$</td>
<td>length of the wall</td>
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<tr>
<td>$L_y$</td>
<td>height of the wall</td>
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<tr>
<td>$M_a$</td>
<td>current analysis moment for calculating $L_e$</td>
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<tr>
<td>$M_{ci}$</td>
<td>moment about the base crack for panel $i$</td>
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<tr>
<td>$M_{cr}$</td>
<td>cracking moment</td>
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<tr>
<td>$\mu_\Delta$</td>
<td>displacement ductility</td>
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<tr>
<td>$\mu_\psi$</td>
<td>curvature ductility</td>
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<tr>
<td>$M_{sj}$</td>
<td>moment about panel base due to shear along yield line $j$</td>
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<td>$M_u$</td>
<td>ultimate moment</td>
</tr>
<tr>
<td>$M_{ux}$</td>
<td>moment capacity about the $y$ axis</td>
</tr>
<tr>
<td>$M_{uy}$</td>
<td>moment capacity about the $x$ axis</td>
</tr>
<tr>
<td>$M_{wi}$</td>
<td>moment about base of panel $i$ due to applied load</td>
</tr>
<tr>
<td>$M_y$</td>
<td>yield moment</td>
</tr>
<tr>
<td>$T$</td>
<td>fundamental wall period</td>
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<tr>
<td>$w_u$</td>
<td>ultimate load on the wall</td>
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<tr>
<td>$w_{uo}$</td>
<td>additional applied load in force-deflection yield line analysis</td>
</tr>
<tr>
<td>$x_0$</td>
<td>projected yield line length in the $x$ direction</td>
</tr>
<tr>
<td>$y_0$</td>
<td>projected yield line length in the $y$ direction</td>
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<td>$\zeta$</td>
<td>hysteretic damping ratio</td>
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1 Introduction

1.1 Background

Unreinforced masonry has been used as a construction material for thousands of years. While some of these structures survive to this day, these tend to have massive walls and would be unpractical for modern use. Modern unreinforced masonry of practical thickness has been shown to perform poorly under out-of-plane loads caused by seismic activity and vertical load eccentricity. In New Zealand, unreinforced masonry is not permitted in active seismic regions by NZS 4230: 1990 and NZS 4229: 1999.

Masonry is a composite material consisting of concrete blocks, grout, mortar and reinforcing steel. It is used because it can be easily constructed with no special equipment, and it is known to be strong and stiff in-plane.

Reinforced masonry can be either partially or fully grouted. The blocks have cavities, and in fully grouted masonry the entire wall is filled with grout once the blocks are laid and steel installed. In partially grouted masonry, only the cavities that contain steel are filled. This saves construction time, lowers grout requirements, and reduces the mass of the wall, consequently reducing seismic loads. This research focussed on the testing and analysis of a partially grouted wall.

While the in-plane strength of reinforced masonry can be estimated from conventional structural mechanics, the out-of-plane strength is harder to determine by simple methods. Several indirect methods of varying complexity and accuracy exist – elastic plate theory, unpropped and propped cantilever models, the strip method, the frame analogy, finite element analysis and yield line theory. However, experimental data is required for the accuracy of these methods to be assessed.

Research on the out-of-plane performance of these walls has mostly been completed in the last 20 years. Of this research, few tests have been done on full-scale models [Clough, 1979; Singh, 1998; Zhang, 1998], with the majority being small-scale tests on simple panels, often with artificially imposed boundary conditions [Drysdale and Essawy,
While these small tests help to obtain the material properties of masonry, they offer little information relevant to the design of the more complicated walls found in practice. More full-scale tests are required to provide practical information for designers and code-writers.

The out-of-plane load resisting mechanism consists of two parts. Firstly there is the simple vertical cantilever action caused by the fixity at the base. This is the primary load resisting mechanism at small loads and deflections. As the loads become larger and the wall deflects more, cracks open along the base of the wall, limiting its fixity. The bond-beam along the top two masonry courses now begins to deform and contribute to load resistance, providing a strong link spanning between the return walls. The wall now exhibits propped cantilever behaviour, with limited fixity at the base and a point load at the top. This model of a propped cantilever can be used to obtain a conservative estimate of wall strength for a wall without openings.

1.2 Research Scope

The project consisted of the construction and testing of one full size 9.4m long by 2.4m high partially grouted masonry wall with 1.8m long return walls at the end, as shown in Figure 1.1. The wall was constructed on a Firth Industries Ribraft flooring system, a 9.4m long, 3.6m wide 305mm deep waffle slab. The reinforcing details used were the minimum or in some cases, less than minimum required by the current non-specific masonry design standard, NZS 4229: 1999. Openings were located such as to reduce the out-of-plane capacity of the wall considerably – one at the wall corner, one diagonally along a 45° yield line.

The wall was loaded out-of-plane using an air pressure system to simulate seismic loading and load-deflection response was recorded in the elastic and post-elastic regions. These were compared with theoretical predictions by a variety of analysis techniques. This was used to determine the accuracy of these theoretical tools, and to determine the degree of conservatism in the non-specific design code NZS 4229: 1999 for this specific wall configuration.
1.3 Research Objectives

- To study ultimate load capacity and failure mechanism
- To evaluate the effectiveness of the reinforcing details, especially the 90° angled bar above the corner window
- To investigate the effect of the openings on load-deflection response, and the stiffness degradation under cyclic loading
- To study the distribution of loading and propagation of cracking in the return walls
- To compare the results of theoretical predictions of load capacity and deflection with the experimental results, and thereby to determine the accuracy of the various theoretical methods
2 Literature Review

2.1 Experimental Testing Programmes

A large proportion of the research on the out-of-plane behaviour of masonry walls focuses on test specimens that are small and have artificial boundary conditions, and the majority of the available literature reflects this. However, there is still a considerable volume of information available relevant to this project.

Drysdale et al [1988] tested 21 full-scale walls under uniform pressure loading normal to the face of the wall. Variable lengths of wall were tested, with different combinations of boundary conditions (simply supported, fixed or free). Tests were also completed on specimens precompressed by vertical loading. They found that the initial cracking load was highly variable, and suggested that this was due to the indeterminacy of the crack patterns. Finite element analysis and yield line analysis were used to predict the failure load of each specimen, with considerable success.

Research by Scrivener [1969] on four reinforced hollow-brick walls found that the mortar strength had an influence on the cracking load, but had no effect on the failure load of the wall.

Adham et al [1990] tested nine full-scale (6m and 7.5m high) masonry walls at the University of California, Berkely. The test was designed to examine the out-of-plane response of the walls away from corners, which tend to improve load-carrying capacity and stiffness. The height, vertical load eccentricity, grouting, quantity of vertical reinforcing and location and number of lap splices were varied. The walls were loaded dynamically with a scaled earthquake excitation at the base and simulated diaphragm effects at the top.

The test program found the walls displayed no visible damage up to an effective peak acceleration of 0.4g. At an EPA of 0.8g, wall mid-height deflections corresponding to drifts of 4% to 6% were observed. The walls were left with permanent offsets of 100mm to 150mm at the end of shaking, although they were still strong enough to support the imposed vertical eccentric loads and their own weight. Yielding of the reinforcing steel
at midheight was noted. It was noted that the presence or absence of previous damage could influence wall behaviour, especially stiffness, by a factor of up to 5.

Abboud et al [1996] tested 6 vertically spanning masonry walls under out-of-plane monotonic loading. Increasing the amount of vertical reinforcing was found to increase the ultimate load capacity but reduce the displacement ductility. Cracking load was found to be unaffected by vertical reinforcing level. The extent of grouting was shown to affect the cracking load, permissible extreme fibre tensile stress and flexural rigidity.

Sveinsson et al [1988] conducted dynamic testing into the inelastic range. This was completed because it was felt that static monotonic or cyclic tests were not accurately representing the effects of a strong earthquake on a wall. These dynamic tests showed that the walls could sustain large deflections with cracking at the base joint being the only major damage. The walls showed large permanent offsets at the end of testing.

A five-storey masonry building was tested at University of California, San Diego. Based on this testing Priestly [1994] suggests that design practices are conservative, as while out-of-plane movement was recorded, overall wall stability was not considered to be a problem.

2.2 Analytical Modelling Research

Sveinsson and Kelly et al [1988] developed non-linear flexural elements for the analysis of centrally reinforced masonry. These elements were intended to model the out-of-plane behaviour past the elastic range. A testing program was carried out to provide data for comparison with the analytical model, and the results were found to correlate strongly. From this, it was determined that their elements provide an accurate prediction of a wall’s behaviour.

Essawy and Drysdale [1988] compiled a summary of the available analysis tools. These tools’ performance was compared to test data and finite element analysis results. It was determined that yield line analysis provided a “reasonably accurate capacity prediction". It was noted that as masonry is such a brittle material, lines of cracking may be simply the
progress of a crack along the wall face and not a line of constant moment. Other simpler
techniques were investigated, and were found at be reasonably accurate if the wall was
simple and suitable boundary conditions could be applied.

Seah, Dawe and Dukuze [1993] developed a computerised approach to the analysis of
masonry walls. The program develops a finite element model of the wall and uses this
model to determine where likely locations for initial cracking are. These cracking
locations are input into a yield line analysis, which gives a value for the ultimate load.
The yield line pattern is then varied iteratively to obtain the lowest ultimate load capacity,
which will be the most accurate answer. This system was used to analyse 90 masonry
walls intended for a nuclear power plant, some of which had complicated openings and
boundary conditions. The final solution for these complicated panels was often quite
different from that a designer might estimate to be correct, hence this system provides a
good check, as the lowest solution in yield line analysis is closest to the true solution.
The calculated strengths of the panels were found to be in excess of that required by the
current codes and standards.

Abboud et al [1993] determined that under inelastic behaviour, deflections of masonry
walls may be critical. A study of the current methods for deflection estimation was
completed and it was found that all methods usually gave too small deflections before
cracking and much too large deflections post-elastic. It was determined that no available
method was sufficiently accurate to be used for design.

2.3 Recent Research at Canterbury

Shivas Singh [1998] completed research on the out-of-plane behaviour of a partially
grouted masonry wall with no openings. This research aimed to find the capacity of the
wall, and also to validate the Ribraft flooring system. A 9.4m long by 2.4m high wall
was constructed with 2.5m long return walls. The outcome of this research was that
masonry was found to be considerably stronger cut-of-plane than the code requirements
of NZS 4229: 1986 would suggest. The Ribraft floor performed excellently.
Xudong Zhang [1998] finished his Master’s research on the behaviour of masonry walls with openings. He tested two specimens – one with door openings at both ends and the other one with a door opening at one end, a corner window at the other, and a window located centrally. Both specimens retained the same overall dimensions as Singh’s work, with the exception of the return wall length being reduced to 1.8m. It was found that the wall openings had a considerable influence on the ultimate load capacity of the walls.

![Diagram](image)

**Figure 2.1 – Recent research specimens at University of Canterbury**

Zhang also evaluated several existing methods of analysis of masonry walls, namely:

- The Cantilever Model
- The Propped Cantilever Model
- The Strip Method
- The Frame Analogy
- Finite Element Analysis
- Yield Line Theory

Of these methods, only the finite element analysis and the yield line theory can be accurately adapted to the analysis of walls with openings. A combination of these two methods will be used in this project. Yield line theory can only give a value for ultimate
load, and for this was found to give a reasonably accurate but conservative estimate. Finite element analysis was found to give accurate results at low levels of load and deflection, but past cracking gave conservative load carrying capacities and a failure deflection many times smaller than that found experimentally.

2.4 NZS 4229: 1999 (Non-specific design code)

NZS 4229: 1999 is the code governing the non-specific design of masonry buildings in New Zealand. As the code does not require specific design, there is a set of regulations governing its use:

- The building must be founded on suitable foundations.
- The building must be under 10m in total height, and no storey may be over 3m.
- The building height to minimum building width ratio must be less than 2.5.
- The plan area must be under certain specified requirements.
- The building must be category IV or V. This means the building must not be expected to house crowds or contents of a high value to the community. Its loss must not have a severe impact on society.
- There are restrictions on allowable attached light-weight components (roofs, suspended floors etc) and their characteristics.

It provides two different methods of out-of-plane load resistance – continuous support by a diaphragm, or support by a reinforced bond-beam spanning between return walls at the top of every storey.

NZS 4229: 1999 was revised from NZS 4229: 1986, partly due to the findings of Singh’s and Zhang’s research. Three major changes were made. The use of partially filled masonry in all earthquake zones is now permitted. The permissible bond-beam spans between return walls were increased. The rules governing the use of concrete floor diaphragms were modified.
The code also specifies requirements on the different material components of the wall, and on the level of workmanship required. The wall is required to be built by a “suitably qualified tradesperson”, and inspection must be completed by a registered mason.

2.5 Overview of Present Research

This report covers the construction and testing of the masonry test specimen, and the subsequent data analysis and comparison with theoretical predictions. It is arranged in the following manner:

The research project is introduced followed by a description of the scope and objectives of the current research.

This is followed by a literature survey on the topic with attention paid to both experimental and theoretical research. Recent research at the University of Canterbury is outlined, and NZS4229: 1999 reviewed.

A description of the test specimen is given with construction methodology, material properties and design information.

Techniques for predicting the out-of-plane behaviour of a masonry wall, focussing on finite element analysis and yield line analysis, are outlined.

The apparatus that was used to load the wall and the instrumentation that was used to record its behaviour is described.

Test results and observations made during testing are displayed.

Test results are discussed and summarises, with important points noted.

Conclusions are drawn from the results of the research programme.

Suggestions are made for beneficial future research.
Appendix 1 contains detailed plans of the test specimen.

Appendix 2 shows the layout of instrumentation used in the experiment.

Appendix 3 contains a record of the crack progression through the loading cycles.

Appendix 4 contains data and plots from the finite element analysis of the test specimen.

Appendix 5 provides a photographic record of the construction, testing and demolition of the test specimen.
3 The Test Specimen

The test specimen was constructed in the Structures Laboratory of the Civil Engineering Department at the University of Canterbury. At various stages of construction, photographs were made of the progress of the wall. Rather than include these photographs in the main text, they have been compiled as Appendix 5.

3.1 The Ribraft Floor

The flooring system used was a Firth Industries Ribraft floor. This floor was validated for this type of masonry construction as part of Singh’s research. It consists of a 305mm thick concrete slab that contains 1100 x 1100 x 220mm polystyrene blocks. Between these blocks are 100mm gaps in which a HD12 bar is suspended with a specially designed hanger. Around the perimeter of the floor there is a 300mm wide beam containing 3 HD12 bars. A diagram of the floor has been included as Figure 3.1. Under the base of the wall, a 600mm wide beam was poured. This was made this wide so there would be room to construct another test specimen if desired on the same floor after the completion of this research.

Figure 3.1 – Detailed layout of pods and spacers
Source: Firth Industries Ribraft Floor System Manual
The concrete used was the specially designed Raftmix Pump, produced by Firth Certified Concrete. It has a target 28 day strength of 20 MPa and a target slump of 120mm. The floor required two truckloads – the first had a slump of 200mm, and the second had a slump of 120mm. The concrete strengths at 7 days, 28 days and start of testing as determined using the Civil Engineering Department’s Avery test machine are as follows:

Table 3.1 – Floor concrete cylinder compressive test results

<table>
<thead>
<tr>
<th>Date Tested</th>
<th>Specimen Age (days)</th>
<th>( f'_c ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/11/00</td>
<td>7</td>
<td>19.0</td>
</tr>
<tr>
<td>9/11/00</td>
<td>7</td>
<td>15.9</td>
</tr>
<tr>
<td>9/11/00</td>
<td>7</td>
<td>15.8</td>
</tr>
<tr>
<td>30/11/00</td>
<td>28</td>
<td>19.7</td>
</tr>
<tr>
<td>30/11/00</td>
<td>28</td>
<td>22.7</td>
</tr>
<tr>
<td>30/11/00</td>
<td>28</td>
<td>24.7</td>
</tr>
<tr>
<td>17/01/01</td>
<td>76</td>
<td>24.2</td>
</tr>
<tr>
<td>17/01/01</td>
<td>76</td>
<td>20.5</td>
</tr>
<tr>
<td>17/01/01</td>
<td>76</td>
<td>20.1</td>
</tr>
<tr>
<td>17/01/01</td>
<td>76</td>
<td>21.8</td>
</tr>
</tbody>
</table>

The concrete in the floor achieved the target strength before 28 days. During testing no damage to the floor was observed around the wall base or load frame attachments.

Hooked D12 starter bars were originally to have been installed with the floor steel before the concrete pour, but it was determined that this would greatly interfere with levelling and screeding the floor. Instead, the floor was poured without starter bars and allowed to harden. 14mm diameter holes were drilled 275mm into the floor at the starter bar locations, and the D12 starter bars were fastened in place with Ramset Ultra-Fix Plus. The starter bars were examined at the end of testing and no pull-out was determined to have occurred.
3.2 The Masonry Wall

3.2.1 Masonry Blocks

The wall was constructed of 20-series blocks obtained from Firth industries. The blocks are formed from a very dry cement/water/sand mix that is placed in a mould and compressed by a hydraulic ram. Six blocks were randomly selected from the pallets used to build the wall and returned to Firth Industries for compressive strength testing. The blocks were tested in accordance with AS/NZS 4456.4: 1997 with the following results:

Table 3.2 – Masonry block compressive test results

<table>
<thead>
<tr>
<th>Date Tested</th>
<th>Block Age (days)</th>
<th>$f'_{cb}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24/01/01</td>
<td>185</td>
<td>30.2</td>
</tr>
<tr>
<td>24/01/01</td>
<td>176</td>
<td>30.8</td>
</tr>
<tr>
<td>24/01/01</td>
<td>223</td>
<td>32.2</td>
</tr>
<tr>
<td>24/01/01</td>
<td>176</td>
<td>33.0</td>
</tr>
<tr>
<td>24/01/01</td>
<td>223</td>
<td>33.9</td>
</tr>
<tr>
<td>24/01/01</td>
<td>241</td>
<td>35.3</td>
</tr>
</tbody>
</table>

Average block compressive strength was taken to be $f'_{cb}=32.6$MPa. The blocks were laid over two days by Anderson Blocklaying Ltd in accordance with NZS 4210: 1989 and standard industry practices. The beam above the windows was temporarily braced with 100x50mm timber and plywood. Inspection ports were installed at the base of every vertical cell that was to be filled with grout, and plywood panels wired tightly over these ports.

3.2.2 Mortar

A carefully measured 4:1 cement:water mix was used to mortar the blocks together. Mortar test cylinders were taken from each batch. These cylinders were cured in the fog room and when tested in the Avery testing machine gave the mortar compressive strength to be:
Table 3.3 – Mortar cylinder compressive test results

<table>
<thead>
<tr>
<th>Date Taken</th>
<th>Date Tested</th>
<th>Specimen Age (days)</th>
<th>$f'_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16/11/00</td>
<td>23/11/00</td>
<td>7</td>
<td>6.3</td>
</tr>
<tr>
<td>16/11/00</td>
<td>23/11/00</td>
<td>7</td>
<td>6.5</td>
</tr>
<tr>
<td>20/11/00</td>
<td>27/11/00</td>
<td>7</td>
<td>5.1</td>
</tr>
<tr>
<td>20/11/00</td>
<td>27/11/00</td>
<td>7</td>
<td>5.8</td>
</tr>
<tr>
<td>16/11/00</td>
<td>14/12/00</td>
<td>28</td>
<td>6.7</td>
</tr>
<tr>
<td>16/11/00</td>
<td>14/12/00</td>
<td>28</td>
<td>8.8</td>
</tr>
<tr>
<td>20/11/00</td>
<td>18/12/00</td>
<td>28</td>
<td>9.1</td>
</tr>
<tr>
<td>20/11/00</td>
<td>18/12/00</td>
<td>28</td>
<td>9.3</td>
</tr>
<tr>
<td>16/11/00</td>
<td>18/01/01</td>
<td>70</td>
<td>7.7</td>
</tr>
<tr>
<td>16/11/00</td>
<td>18/01/01</td>
<td>70</td>
<td>9.8</td>
</tr>
<tr>
<td>20/11/00</td>
<td>18/01/01</td>
<td>66</td>
<td>10.1</td>
</tr>
<tr>
<td>20/11/00</td>
<td>18/01/01</td>
<td>66</td>
<td>13.1</td>
</tr>
</tbody>
</table>

Once the blocks had been laid, the mortar joints were tooled with a 15mm tool to provide an acceptable finish. Any mortar that began to dry out was discarded rather than permit any addition of water and remixing.

3.2.3 Reinforcing Steel

Steel was installed in the wall in accordance with the minimum provisions of NZS 4229:1999. This gave D12 bars vertically every 800mm and under the windows, and D16 bars running horizontally in the bond beam. All reinforcing steel was obtained from Fenwick Reinforcing Ltd. Steel layout can be seen in more detail in Appendix 1. Samples were taken from various bars that were installed in the wall, and their yield and ultimate strengths and strains were identified using the Avery testing machines as follows:
Table 3.4 – Reinforcing steel tensile test results

<table>
<thead>
<tr>
<th>Bar Type</th>
<th>Location</th>
<th>$f_y$ (MPa)</th>
<th>$\varepsilon_y$ (%)</th>
<th>$\varepsilon_{sh}$ (%)</th>
<th>$f_u$ (MPa)</th>
<th>$\varepsilon_u$ (%)</th>
<th>E (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D12</td>
<td>Starter Bar</td>
<td>300.0</td>
<td>0.16</td>
<td>3.6</td>
<td>408.6</td>
<td>21.7</td>
<td>192</td>
</tr>
<tr>
<td>D12</td>
<td>Starter Bar</td>
<td>302.3</td>
<td>0.18</td>
<td>3.4</td>
<td>408.6</td>
<td>21.6</td>
<td>170</td>
</tr>
<tr>
<td>D12</td>
<td>Vertical Bar</td>
<td>317.6</td>
<td>0.13</td>
<td>3.5</td>
<td>410.0</td>
<td>23.3</td>
<td>242</td>
</tr>
<tr>
<td>D12 mean</td>
<td></td>
<td>306.7</td>
<td>0.16</td>
<td>3.5</td>
<td>409.1</td>
<td>22.2</td>
<td>201.4</td>
</tr>
<tr>
<td>D16</td>
<td>Bond Beam</td>
<td>313.4</td>
<td>0.18</td>
<td>3.3</td>
<td>452.7</td>
<td>20.3</td>
<td>178</td>
</tr>
<tr>
<td>D16</td>
<td>Bond Beam</td>
<td>323.4</td>
<td>0.15</td>
<td>3.6</td>
<td>454.7</td>
<td>22.4</td>
<td>212</td>
</tr>
<tr>
<td>D16</td>
<td>Corner Detail</td>
<td>314.4</td>
<td>0.15</td>
<td>3.5</td>
<td>456.5</td>
<td>23.4</td>
<td>211</td>
</tr>
<tr>
<td>D16</td>
<td>Corner Detail</td>
<td>328.4</td>
<td>0.17</td>
<td>3.4</td>
<td>465.2</td>
<td>22.1</td>
<td>196</td>
</tr>
<tr>
<td>D16 mean</td>
<td></td>
<td>319.9</td>
<td>0.16</td>
<td>3.4</td>
<td>457.3</td>
<td>22.1</td>
<td>199.3</td>
</tr>
</tbody>
</table>

The variability in $E$ can be attributed in part to there only being 12 increments of displacement measurement out to yield. Apart from this, the steel showed considerable consistency, as can be seen in Figure 3.2.

![Stress-strain curves for D12 steel specimens](Figure 3.2)
Steel details were in some instances different from those specified in NZS 4229: 1999. The bars running under the window are supposed to run straight for 640mm, but in the test specimen they were hooked downwards for 150mm into the cells directly beside the window. This simplified the grouting process.

The detail at the intersection of the return wall and main wall bond beam was modified. Previous research [Zhang, 1988] found the codified detail to be weak when an opening moment was applied. In light of this, the “clip-bar” detail was used which provides tensile steel to resist this opening.

![Diagram of NZS 4229: 1999 suggested detail and detail used in test specimen.](image)

**Figure 3.3 – Bond beam intersection details**

3.2.4 Grout

The wall was to be partially grouted, meaning that only the cells that contained reinforcing steel were filled. A 17/13 grout was obtained from Firth Certified Concrete, which had a target 28 day strength of 17.5 MPa. The largest aggregate in the grout was 13mm and the spread was 350mm. This finely graded fluid concrete was ideal for pumping and getting down the narrow spaces between the rebar and the blocks.

The cells were 1/3 filled then rodded repeatedly. This process was repeated twice until the cells were completely filled. Grout spilled slightly out of the inspection ports at the bottom of the filled cells. This provided confirmation that the grout had reached the
bottom of the cells, which is very important as the grout provides the bond between the starter bars and the vertical wall steel. After previous research, the walls had been smashed open during demolition to reveal large voids inside the wall, so extra care was taken to ensure complete grout penetration. Because of this, when the wall was finally demolished it was found that the vertical cells were completely filled.

Grouting was completed on 27/11/00, with all cylinders being cured in the fog room. Due to the timing of the grout pour and the Christmas holidays, no 28 day strength was obtained. The tests, performed on the Avery test machine gave the following strengths:

<table>
<thead>
<tr>
<th>Date Tested</th>
<th>Specimen Age (days)</th>
<th>$f'_g$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4/12/00</td>
<td>7</td>
<td>11.5</td>
</tr>
<tr>
<td>4/12/00</td>
<td>7</td>
<td>11.3</td>
</tr>
<tr>
<td>4/12/00</td>
<td>7</td>
<td>11.5</td>
</tr>
<tr>
<td>8/01/01</td>
<td>42</td>
<td>18.3</td>
</tr>
<tr>
<td>8/01/01</td>
<td>42</td>
<td>17.2</td>
</tr>
<tr>
<td>8/01/01</td>
<td>42</td>
<td>18.6</td>
</tr>
<tr>
<td>17/01/01</td>
<td>51</td>
<td>17.1</td>
</tr>
<tr>
<td>17/01/01</td>
<td>51</td>
<td>19.6</td>
</tr>
</tbody>
</table>

Grout strength for analysis was taken to be the 51 day average, $f'_g$=18.6MPa. This is just above the target strength of 18.5 MPa at the time of testing, and it is unlikely that the grout met this strength after 28 days.

3.2.5 Masonry Composite Properties

Masonry is a composite material, consisting of blocks, grout and mortar. The failure mechanism of masonry under compression is vertical splitting of the blocks. This is caused by the mortar’s high poisson’s ratio and relatively low strength, which causes it to
squeeze outwards under load. The blocks attempt to resist this spreading, but have a low tension capacity and split vertically.

The strengths of the separate materials were determined separately with simple compressive tests, and these strengths combined to give a combined masonry strength, $f'_{m}$. This is given in NZS 4230:1990 in Appendix C (Equation C-1) as

$$f'_{m} = 0.45\alpha f'_{ch} + 0.675(1-\alpha)f'_{g}$$

Where

- $\alpha$ = maximum ratio of net block area to gross masonry unit area
- $f'_{ch}$ = average block compressive strength
- $f'_{g}$ = average grout strength

Using an $\alpha$ ratio of 0.53, and the material properties from the test results, the calculated effective masonry compressive stress $f'_{m} = 13.7$ MPa.

For the purposes of determining material properties, a width of strip to be analysed had to be determined. For the bond beam, the full height of 400mm was used. For the vertical strip, the width of masonry with 1 D12 bar, or 800mm, was used. For the unreinforced horizontal strip, a nominal 1000mm wide strip was selected.

The value for the modulus of rupture for the bond beam, vertical strip and horizontal strip are determined empirically.

<table>
<thead>
<tr>
<th>Location</th>
<th>Equation</th>
<th>$f_{cr}$ (MPa)</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal Strip</td>
<td>$0.21\sqrt{f'_{m}}$</td>
<td>0.78</td>
<td>UBC-94</td>
</tr>
<tr>
<td>Vertical Strip</td>
<td>$0.17\sqrt{f'_{m}}$</td>
<td>0.63</td>
<td>UBC-94</td>
</tr>
<tr>
<td>Bond Beam</td>
<td>$0.63\sqrt{f'_{m}}$</td>
<td>2.33</td>
<td>Park [1975]</td>
</tr>
</tbody>
</table>
First cracking was assumed to occur when this stress was reached on the tension side of the masonry strip. First yield was where the steel at the mid-height of the strips reached its yield strain. Ultimate strength was taken when the fibre at the compressive edge had reached a strain of 0.003.
4 Testing Apparatus

4.1 The Loading Frame

The loading frame used was constructed for Singh’s research and modified for Zhang’s. For the current research, the frame was further modified by the addition of RHS steel sections to the loading face to replace the timber previously used. To these RHS sections, 18mm plywood was attached to provide a smooth face for the airbags to bear against.

36mm diameter high strength threaded rods were grouted into the Ribraft floor. The frame was attached to these protruding rods with high strength nuts. This prevented the front of the frame lifting up. A steel angle was attached to the floor behind the frame. By blocking back to this beam the frame could be prevented from sliding backwards. However, it was found that at low levels of load this was unnecessary, as the bearing of the base of the reaction frame on the floor was sufficient to prevent sliding. An end elevation of the testing rig has been included as Figure 4.1.

Figure 4.1 – The reaction frame and airbag system
4.2 The Airbag System

The airbags were originally designed for use as a packing system in shipping containers. They are inserted with cargo and inflated to prevent the cargo shifting during transportation.

As the airbags had been used in previous research, some of them had taken considerable damage. This necessitated the testing of each airbag to ensure that it could take the predicted ultimate load that the wall might resist, and that it could take this load without leaking air at an unacceptable rate. Airbags with obvious damage were repaired with tape, and testing was accomplished by placing them under a length of hollow-core prestressed floor that was being stored for other research and inflating to 10kPa.

Once the integrity of the airbags was established they were attached to the plywood facings of the loading frame. A high-flow manifold was designed to allow the even distribution of air throughout the loading system, which has been represented schematically as Figure 4.2. A similar manifold had been used in previous testing with manometers set up to record the pressure differential along the length of the wall. As this had been found to be negligible, instrumentation to check this was not installed.

![Figure 4.2 – Air pressure system showing airbag take-off locations.](image)

If the bags were allowed to bulge through a window opening, there was a high chance of them bursting at the pressures expected at the end of testing. Because the loading system had to work on both faces of the wall, valves were installed to allow different combinations of airbags to inflate, with the bags that were to overlap windows being shut...
off. This did result in a reduced loading area, as the areas above the window on that airbag had no pressure applied; the area loaded can be seen in Figure 4.3.

![Figure 4.3 – Area of wall loaded by the airbags](image)

4.3 Instrumentation

Various forms of instrumentation were used to measure and record deflection, strain in steel and masonry, loading pressure, temperature and crack size. All data for pressure, deflection and strain was recorded continuously with computerised data logging system.

4.3.1 Electronic Strain Gauges

61 type FLA-5-11 foil electronic resistance strain gauges were installed in quarter-bridge configuration on wall steel at various locations. These were on the starter bars, around the windows, at wall mid-height and in the bond beam. Appendix 2 gives location details. Each gauge was orientated to measure the longitudinal strain in the steel, with a maximum accurate strain of 2%.

The gauge locations had to be filed free of deformations then sanded smooth with emery paper. The location was then cleaned repeatedly with methyl ethyl ketone until no trace of discolouration could be seen on the cleaning swab. The gauge was then attached with Loctite 401 fixing compound, taking care not to touch with hands the cleaned surface or the gauge mounting face at any stage. 5 layers of protective rubber insulating/waterproofing compound were then applied, and the entire assembly wrapped with a thick rubber tape to prevent damage. This protection worked so well that no strain gauges were damaged during grouting, with all reading the desired 120Ω when tested afterwards.
From previous research [Singh, Zhang], it was found that environmental conditions, especially temperature, could alter the readings given by these strain gauges considerably – enough so that test specimens constructed outside had to be provided with temporary shelter. As this research specimen was constructed inside the Civil Engineering Laboratory, in a location chosen to prevent direct sunlight falling on the project as far as possible, these effects were minimised and determined unnecessary to evaluate.

4.3.2 Potentiometers

13 potentiometers were installed on the test specimen. Of these eight were measuring the out of plane deflection along the bond beam. Five were measuring the out-of-plane deflection of the two return walls. The potentiometers were spring loaded to bear against small plates attached to the top of the wall, as shown in Figure 4.4. As these plates were 50mm higher than the wall, and the wall effectively rotates about it’s base, the deflections measured were reduced by 2% to obtain the true deflection of the top of the wall.

Figure 4.4 – Potentiometer tip bearing onto plate fixed to top of wall

The use of a relatively low number of potentiometers on a large specimen was justified by analysis of the deformation data of previous projects and determining what information is redundant. For example, solid return walls were found to always essentially rotate as a
rigid unit, and required only two potentiometers to uniquely specify their entire deformation pattern.

The potentiometers varied in size from 300mm travel down to 50mm travel. Previous research was used to determine the maximum likely deflection at each measurement site, as to increase accuracy it was desirable to use the smallest potentiometers possible at each site. This was because the potentiometers have 4000 increments of deflection regardless of their size, so a 300mm potentiometer has a measurement increment of \( \frac{300}{4000} = 0.075 \text{mm} \), while a 50mm has a measurement increment of \( \frac{50}{4000} = 0.0125 \text{mm} \).

4.3.3 DEMEC gauge

A DEMEC gauge was used to record strains along the top of the wall and in the return walls during each loading cycle. The gauge had a length of 500mm and could read from 498mm to 504mm, or -0.004 to +0.008 strain. It was constructed to store up to 500 readings, and recorded the temperature as each reading was taken. When these readings were uploaded to a computer, the software that reads the information automatically adjusts for temperature effects. The data is then converted to a spreadsheet file, where the data could be accessed. The strains on the wall faces could then be converted into neutral axis strains, curvatures, shear strains and torsion strains.

Small metal discs with an indentation on one side were attached to the wall with sealing wax. A 500mm spiked bar was placed in these indentations and the wax heated with a small gas torch to allow the discs to slide so the indentations were exactly under the spikes. This gave a gauge length very close to 500mm, which was then recorded before testing began to gave a baseline to measure changes against.

4.3.4 Air Pressure Sensors

An electronic air pressure sensor was used to record the load applied to the face of the wall along with the deflections and strains on the data logging system. It had a pressure range of -5 psi to 5 psi, which was far greater than was required for this testing.
This was backed up with two manual systems. The first was a dial pressure gauge installed at the same point as the flow regulating valve, but this had limited precision. The second was a water manometer that was also used to calibrate the two previous sensors. The layout of these sensors on the airbag system can be seen in Figure 4.2.

4.3.5 Data Logging System

A 133MHz Pentium was used as a data logging system to constantly record air pressure, deflections and strain gauge readings. These were written to a single file for each loading half-cycle. Five purpose-built instrumentation boxes linked in series processed strain gauge information, with the size of the wall rather than the number of channels available governing the number of boxes required. Potentiometer and pressure sensor readings were processed by two general purpose boxes. Only one channel had to be calibrated on each of the strain gauge boxes by connecting a known resistance and comparing this with the reading. Each potentiometer channel and to be calibrated separately to give the correct gain and range. This was accomplished by deflecting the potentiometer a known amount several times, and using a best-fit curve to find the relationship between resistance and deflection.

All computerised calibration information was stored in a .cfg file which allowed testing to be interrupted without having to recalibrate the instrumentation.

4.3.6 Crack Detection and Measurement

The test specimen was painted with a single layer of white undercoat to facilitate the early location and measurement of cracks. When cracks were small, they were measured with a feeler gauge, accurate down to 0.03 mm, and a magnifying lens system marked in 0.02mm increments. Late in the testing regime, the major cracks along the base of the wall grew so large that measurement with a feeler gauge was unnecessary, and crack width was instead measured with a ruler marked in 1mm increments. Cracks were highlighted with marker pens and the crack pattern photographed at each loading increment.
5 Theoretical Studies and Predicted Response

There are several common methods of analysis for determining the serviceability and ultimate strengths of a masonry wall. Of these Elastic Theory, the Hillerborg Strip Method, and the Frame Analogy were all shown to be overly conservative by Zhang [1998] and will not be considered here. These methods are also incapable of dealing with a wall with openings. The two most accurate methods, Finite Element Analysis and the Yield Line Method, will be examined in an attempt to predict the pre- and post-elastic behaviour of the test specimen. These theoretical predictions will be compared with actual test results to determine their accuracy.

5.1 Masonry Element Properties

For use in finite element and yield line analyses, the material properties of the masonry unit must be determined. Finite element analysis requires information on stiffness before and after yield, as well as the curvature at which yield and failure occurs. Yield line analysis requires the level of ultimate moment capacity. To obtain these, moment-curvature analysis will be used.

Depth to steel centroid, d, was taken in all cases to be 95mm. No axial load was assumed, hence the compression block depth \( a = \frac{A_s f_y}{0.85 f_m'} b \) and the ultimate moment capacity \( M_u = A_s f_y (d-a/2) \). Table 5.1 outlines the section properties and the moment capacities of the masonry strips used in analysis. Table 5.2 shows moment of inertia and curvature of the strips at critical points along the \( M-\varphi \) path.

<table>
<thead>
<tr>
<th>Element</th>
<th>b (mm)</th>
<th>( A_s ) (mm(^2))</th>
<th>( f_{cr} ) (MPa)</th>
<th>a (mm)</th>
<th>( M_{cr} ) (kN.m)</th>
<th>( M_y ) (kN.m)</th>
<th>( M_u ) (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bond Beam</td>
<td>400</td>
<td>402</td>
<td>2.33</td>
<td>25.9</td>
<td>5.5</td>
<td>9.7</td>
<td>9.9</td>
</tr>
<tr>
<td>Horizontal Strip</td>
<td>1000</td>
<td>0</td>
<td>0.78</td>
<td>0</td>
<td>3.3</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Vertical Strip</td>
<td>800</td>
<td>113</td>
<td>0.63</td>
<td>3.6</td>
<td>2.4</td>
<td>3.2</td>
<td>3.2</td>
</tr>
</tbody>
</table>

Table 5.1 – Section Properties (1)
Table 5.2 – Section Properties (2)

<table>
<thead>
<tr>
<th>Element</th>
<th>$I_e$ (x10^8 mm^4)</th>
<th>$I_{cr}$ (x10^6 mm^4)</th>
<th>$\varphi_{cr}$ (x10^3 rad/m)</th>
<th>$\varphi_y$ (x10^3 rad/m)</th>
<th>$\varphi_u$ (x10^3 rad/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bond Beam</td>
<td>2.2</td>
<td>25.8</td>
<td>1.79</td>
<td>34</td>
<td>98</td>
</tr>
<tr>
<td>Horizontal Strip</td>
<td>4.1</td>
<td>-</td>
<td>0.60</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Vertical Strip</td>
<td>3.6</td>
<td>9.2</td>
<td>0.48</td>
<td>242</td>
<td>699</td>
</tr>
</tbody>
</table>

The bond beam and vertical strip both have a curvature ductility $\mu \phi$ of 2.9. This would suggest a very low displacement ductility $\mu \Delta$. As test specimens have been shown to exhibit considerably more displacement ductility, this suggests that the assumption that plane sections remain plane may not be valid for reinforced masonry. This will be used in section 5.5 in an analysis that models the behaviour of masonry at large deflections.

The elastic modulus of the masonry was taken as $E_m=1000\Gamma_m$ [Paulay 1992] and this combined with $I_e$ and $I_{cr}$ gives an indication of the pre- and post-cracked stiffness of the strips. Figure 5.1 and Figure 5.2 show the $M$-$\phi$ paths of the masonry strips.

![Figure 5.1 – Moment-curvature relationship for Bond Beam](image-url)
Figure 5.2 – Moment-curvature relationship for Vertical and Horizontal Strips

It can be seen that the cracking curvatures are very small compared with the yield and ultimate curvatures. This agrees with previous experimental results, in which the walls were found to crack at very low deflections when compared with their ultimate deflection capacity.

A more refined analysis could be completed by breaking the masonry strips down into many thin layers and using the constitutive relationships for the individual materials to solve for the moment at a given curvature. This would obtain every point along the M-\( \phi \) path, and allow a more accurate model of the test specimen to be constructed. This approach is usually used to model reinforced concrete sections, however it would require some modification to take into account the effect on the blocks of the lateral expansion of the unconfined mortar layers.

5.2 Finite Element Analysis

5.2.1 Traditional 3D Analysis

Finite element analysis in three dimensions at its simplest level consists of a cuboid with a node at each corner. Each of these nodes has deflection variables in the x, y and z
directions. Added accuracy can be obtained by several methods: refining the mesh by adding elements, adding nodes to the existing elements, and adding rotational degrees of freedom to existing nodes.

These elements are then combined together to form a numerical approximation to the object being analysed. A stiffness matrix is formed for the entire object, the necessary loads are applied, and the deformations are calculated. From these deformations, the strain in the material may be evaluated, and using constitutive relationships, the stresses may be found.

The most accurate results are obtained when the aspect ratio of the elements is reasonably close to unity. If this is not the case, the values in the stiffness matrix become too different, and numerical errors begin to affect the answers.

5.2.2 Plate-Shell Analysis

A plate is generally much broader and higher than it is thick, which poses a problem. To reduce computational effort as much as possible, the mesh in the x and y directions needs to be as coarse as possible. However, as the plate thickness in the z direction tends to be small, this results in elements with high aspect ratios, and leads to inaccurate answers.

To counter this problem, plate-shell bending elements were developed. The in-plane deflections at the top and bottom of the plate were combined into two different degrees of freedom – the rotation and elongation of the centre of the plate, as can be seen in Figure 5.3. The end result of this is a stiffness matrix with all terms of reasonably close order, and a considerable reduction in numerical errors. The plate can now be defined in two dimensions, giving a thickness at each node to represent the third dimension. This simplifies data entry and checking considerably.

Plate-shell elements were developed considerably later than other finite elements. This is because deformation patterns to exactly model plate bending without causing discontinuities of rotation along element boundaries are complicated. Continuous triangles were developed, but these had different degrees of freedom along different edges. It was not until a way was found to combine these inconsistent triangles into
quadrilaterals and triangles that had the same degrees of freedom along all sides that plate-shell analysis became accurate and practical.

Thin conventional element has great differences between in-plane and out-of-plane stiffness, causing numerical errors in the analysis. All four corner nodes must be specified.

Plate-shell element replaces the two displacement degrees of freedom at the element faces with one displacement degree of freedom and one rotation degree of freedom at the element centreline. Only the two centreline nodes must be specified.

Figure 5.3 – In-plane degrees of freedom of conventional 3-D element and plate-shell element

Another available tool for use is the plate bending element. These are similar in formulation to plate-shell bending elements, however they do not take into account in-plane shears and principal stresses. In reality, most plates in structural engineering are subjected to in-plane stresses, be it from membrane actions, temperature variations or directly applied forces at the boundaries. As there is no difference between the two types of element from a model definition point of view, the more realistic plate-shell elements should be used.

5.2.3 Non-linear Finite Element Analysis

Plate bending analysis for an elastic, isotropic material requires simply the specification of the plate thickness, the material elastic modulus, and the Poisson’s ratio. From this all stiffness and stress parameters can be calculated for a given loading. However, for a non-linear material like masonry, more complicated information is required.

When masonry or any concrete member cracks it undergoes a change of stiffness. This change of stiffness must be modelled in an analysis if it is to provide accurate answers. One approach, suggested by UBC-94 [Drysdale et al, 1994], is to modify the effective moment of inertia of the section past cracking as follows:

\[ I_e = I_g \left( \frac{M_{cr}}{M_a} \right)^3 + I_r \left( 1 - \left( \frac{M_{cr}}{M_a} \right)^3 \right) \]
Where $M_{cr} < M_a$. If $M_a < M_{cr}$, the section is uncracked and the gross moment of inertia, $I_g$, should be used.

This approach could also be used to model the cyclic behaviour of masonry. If $M_a$ was taken to be the maximum moment in the loading direction, accounting for moments from previous cycles, this would provide a record of the level of cracking at that location. In future loading cycles the element would then exhibit the level of stiffness applicable to its level of cracking, regardless of whether the current moment is high enough to cause that cracking or not. This would allow better modelling of degradation of stiffness under repetitive multi-directional loading.

Non-linear analysis is more computationally expensive than a linear analysis. This is because at each loading increment the moments at every point must be evaluated and the amount of stiffness calculated. These individual element stiffnesses must then be combined into an overall stiffness matrix and a small increment of load applied. If this loading increment is too large, then changes in the model's stiffness may be missed and inaccurate answers obtained.

This variable moment of inertia approach was used by Zhang to evaluate the capacity of his two walls. In this research, another approach will be used, as detailed below.

5.2.4 Finite Element Analysis using ABAQUS

For this research the wall was modelled using the general-purpose finite element analysis package ABAQUS version 5.7. ABAQUS has many widely varied functions – it can complete elastic and inelastic structural analysis, thermal analysis, electrical analysis and pore water pressure analysis, to name a very small selection. It was chosen for this situation because it gives the ability to model concrete and reinforcing bar directly, removing the need to separately calculate many element properties.

The loaded face only of the wall was analysed, with the base and return wall edges being modelled as fully fixed supports. The wall itself was modelled as an isotropic plate,
which is not strictly correct. To correctly model the pre-cracking and post-cracking behaviour of the wall, the element thickness has been modified.

An isotropic plate-shell element is solid with its thickness specified, and it follows that the moment of inertia will be the same in both directions. As masonry has vertical cavities it is orthotropic, with different properties in different directions. The gross moment of inertia $I_g$ has been determined for each strip in Table 5.3, and a weighted average was used to determine the average moment of inertia used for the analysis.

**Table 5.3 – $I_g$ values for different wall components**

<table>
<thead>
<tr>
<th>Component</th>
<th>$I_g$ (mm$^4$)</th>
<th>Length (mm)</th>
<th>$I_g$ per mm</th>
<th>Weighting</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical strip</td>
<td>3.64E+08</td>
<td>800</td>
<td>4.54E+05</td>
<td>0.5000</td>
</tr>
<tr>
<td>Horizontal strip</td>
<td>4.09E+08</td>
<td>1000</td>
<td>4.09E+05</td>
<td>0.0833</td>
</tr>
<tr>
<td>Bond beam</td>
<td>2.23E+08</td>
<td>400</td>
<td>5.57E+05</td>
<td>0.4167</td>
</tr>
</tbody>
</table>

The total weighting for horizontal and vertical strips is ½ each, with the ½ for horizontal being split between the unreinforced horizontal strips and bond beam in the ratio of their heights. The weighted average was found to be $4.93 \times 10^5$ mm$^4$/mm. Using the formula $I_g = \frac{bd^3}{12}$ and substituting $b=1$mm it was calculated that to get the correct average uncracked stiffness the element should be 181mm thick.

Table 5.1 shows the calculation of moment capacities. In this it can be seen that the maximum depth of the compressive block for any strip is 25.9mm. As the flanges of the blocks are 33mm thick, it is reasonable to assume that the compressive stress block and hence the post-cracking behaviour of the masonry strips would be similar to that of a solid concrete slab with identical reinforcing at the same distance from the compressive face. This determines that in the plate-shell element, the reinforcing bars must be located at 95mm from the compressive face, hence 181-95=86mm from the tensile face.
With the thickness and reinforcing location specified, an element mesh was formed. The elements used were 200mm x 200mm, which gave 47 elements along the length of the wall and 12 elements along the height. This has been displayed as Figure 5.4.

Figure 5.4 – The Finite Element mesh used in ABAQUS

The elements were modelled using S4R plate-shell bending elements. The section was described as 181mm thick, with 9 points of numerical quadrature through the thickness of each element specified. This high number of quadrature points allows for more accurate modelling of the progression of cracking through the elements.

The material was specified next. Initially the elastic (uncracked) Young’s modulus and Poisson’s ratio were specified. To allow inelastic modelling, the concrete parameters of yield stress, ultimate stress and plastic strain at ultimate stress were input. Ratios of biaxial and tensile capacities relative to uniaxial are also required.

Parameters to activate tension stiffening were included next. Tension stiffening provides a linear decrease in stress of cracked concrete, and has been shown to correctly model the transfer of energy from the concrete to the steel as cracking occurs. This is very important for a structure with a low steel ratio such as the test specimen.

This completed the definition of the concrete elements. The next section of the input file deals with adding rebar to the specimen.

ABAQUS deals with rebar in shells by smearing it throughout the elements that contain it. The command line is required to specify the element containing the rebar, the cross-
sectional area of each bar, the spacing of the bars and the orientation of the bars. This was done three times – once to add the vertical steel, once for the horizontal steel in the windowsills, and once to define the steel in the bond beam. The reinforcing bars were then linked to the appropriate element sets, which enabled the properties for these elements to be modified.

Finally, the loading scheme was specified. Loading was increased in small increments by ABAQUS until no solution was available, i.e. the structure had collapsed. A maximum load for the analysis of 10kPa was specified, however this was not expected to be and was not reached.

As an indicator of overall structural behaviour, midspan bond beam deflection was plotted against load. This has been plotted as Figure 5.5, along with experimental results from the +4.8kPa cycle.

![Graph]

Figure 5.5 – Finite Element Analysis midspan load-deflection plot

More detailed description of the input, and graphical samples of the output can be found in Appendix 4.
As the initial stiffness of the model is too high, it appears that the calculated thickness of 181mm was too great. Modification of this thickness may result in a better Finite Element model, but due to time limitations this was not investigated in the course of this project. However, it must be noted that the three previous test specimens at Canterbury displayed initial stiffnesses very similar to that shown in the analysis, suggesting that the 181mm thick element may be suitable in some cases.

Because in a Finite Element Analysis it is assumed that plane sections remain plane, it is incapable of modelling the large rotations across individual cracks characteristic of masonry collapse patterns. This resulted in the analysis terminating at a much lower deflection than was found experimentally; the finite element analysis terminated due to lack of solution stability at 17.3mm, while the wall was tested to 230mm.

The analysis showed that the first yielding occurred in the central starter bars and progressed outwards towards the return walls as load and deflection increased. The load resistance due to the starter bars did not increase substantially above 5mm deflection. From this point to failure, the increase in load resisting capacity was due to increasing stresses in the bond beam.

The distribution of moment about a vertical axis and the distribution of bond beam steel stress showed that the bond beam is well below its moment capacity at all levels of load. The only locations at which it comes close to yielding is at the intersections with the return walls, which were fixed connections in the analysis. This suggests that the walls may be adequately strong with less bond-beam steel or longer spacing between return walls. However, as it was found that partially grouted masonry walls are is governed by deflection requirements, the extra flexibility caused by these changes may make the walls impractical from a design point of view.

5.3 Yield Line Analysis

Johansen’s yield line analysis was initially developed for slabs but is valid for walls with low axial loads. It involves equating the internal work done, in the form of plastic hinging in two directions, with the external work done, in the form of load applied multiplied by deflection. This is an upper bound approach, and any ultimate load
capacity found will be greater than or equal to the true upper bound. Hence different patterns of yield lines must be considered, as the wrong pattern will give a non-conservative load capacity. Yield line analysis is only valid for ductile concrete members with light reinforcing. The test specimen meets both of these requirements.

5.3.1 External Work

The wall is assumed to move as a set of rigid bodies rotating about the yield lines. To evaluate the external work on the entire wall, the external work on each panel is evaluated separately, and the results are summed. By specifying a deflection $\Delta$ at one point on the slab, the deflection $\Delta_i$ at the centroid of any rigid element can be found. The load on each element is found by multiplying the area of the element $A_i$ by the ultimate load, $w_u$. Hence the total external work done is found from:

$$\int \int w_u \Delta(x,y) dx dy = \sum_{i=1}^{n} w_u A_i \Delta_i$$

5.3.2 Internal Work

Bending and torsional moments, and shear forces along the yield lines resist the external load on each plate. However, the work done by shear forces and torsional moments will sum to zero across all plates. Thus only the bending moments will contribute to the internal work content of the wall.

The internal work is summed separately in the $x$ and $y$ directions for each segment. It is calculated for each yield line as the ultimate moment capacity per unit width, multiplied by the projected length of the yield line, multiplied by the rotation about the yield line. Figure 5.6 identifies these components for a vertical yield line.
Figure 5.6 – Internal work components for a panel intersection

As can be seen in the above diagram, the notation for yield line analysis is different to that usually used. $M_{ux}$ and $\partial_{x}$ refer to moments and rotations in the plane of the $x$-axis, not about the $x$-axis as would normally be the case.

For concrete floor slabs that typically do not have a reinforcing layout symmetrical about the neutral axis, the positive and negative moment capacities of the slab must be used where applicable to obtain the correct internal work. However, as centrally reinforced masonry has the same positive and negative moment capacity this is not a consideration in the following analyses.

5.3.3 Ultimate Load Capacity

As an example, for a wall with no openings, fixed at the base and simply supported on the two side edges we can postulate the following yield line pattern:
If the upper edge of the middle panel is given a displacement $\Delta$, then the centroidal displacement of each panel can be calculated. External and internal work, determined for each panel separately, are as follow:

**Table 5.4 – External work done on simple wall**

<table>
<thead>
<tr>
<th>Panel</th>
<th>External Work</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABC</td>
<td>$\frac{1}{3} w_u L L_y \Delta$</td>
</tr>
<tr>
<td>BCDF</td>
<td>$\frac{1}{2} w_u (L_x - 2L) L_y \Delta$</td>
</tr>
<tr>
<td>DEF</td>
<td>$\frac{1}{3} w_u L L_y \Delta$</td>
</tr>
</tbody>
</table>
Table 5.5 – Internal work cone on simple wall

<table>
<thead>
<tr>
<th>Panel</th>
<th>$\partial_x$</th>
<th>$\partial_y$</th>
<th>$M_{ux}y_0\partial_x$</th>
<th>$M_{uy}x_0\partial_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABC</td>
<td>$\Delta \over L$</td>
<td></td>
<td>$M_{ux}L_y \Delta \over L$</td>
<td>0</td>
</tr>
<tr>
<td>BCDF</td>
<td></td>
<td>$\Delta \over L_y$</td>
<td>0</td>
<td>$2M_{uy}L \Delta \over L_y + M_{uy}L_x \Delta \over L_y$</td>
</tr>
<tr>
<td>DEF</td>
<td>$\Delta \over L$</td>
<td></td>
<td>$M_{ux}L_y \Delta \over L$</td>
<td>0</td>
</tr>
</tbody>
</table>

By summing and equating external and internal work it is found that

$$2M_{ux}L_y \Delta \over L + 2M_{uy}L \Delta \over L_y + M_{uy}L_x \Delta \over L_y = 2 \over 3 w_u L L_y \Delta + \frac{1}{2} w_u (L_x - 2L)L_y \Delta$$

Solving for $w_u$ gives

$$w_u = \frac{2M_{ux} L_y \Delta \over L + 2M_{uy} L \over L_y + M_{uy} L_x \over L_y}{\frac{2}{3} L L_y + \frac{1}{2} (L_x - 2L) L_y}$$

However, this will still leave the unknown length $L$ in the equation. This is removed by finding the minimum value for $w_u$ for any value of $L$. To obtain this a solution for $\frac{\partial w_u}{\partial L} = 0$ must be found. To simplify this, the substitutions of $L_x = 9.4m$, $L_y = 2.4m$ and $M_{ux} = M_{uy}$ will be made, and it is found that $L = 1.80m$. Substituting this solution into our original expression for $w_u$ and setting $M_{ux} = M_{uy} = 1.0kPa$ find that $w_u = 0.821kPa$.

In a more complicated wall or slab there might be several unknowns such as $L$ that must be solved for. This can be accomplished by solving each differential equation separately and then solving the set of simultaneous equations that result. However, this could quickly become a major undertaking, so Park [1980] suggested and proved that the yield
lines entering the corners of the panel at 45° is a valid assumption. Based on this assumption, \( L_y = L_x \) and the solution for the simple wall would be

\[
w_y = \frac{2M_{ux} + 2M_{oy} + M_{wy} \frac{L_x}{L_y}}{\frac{2}{3}L_y L_y + \frac{1}{2}(L_x - 2L_y)L_y}
\]

To establish the difference between these two expressions for \( w_y \), we can again substitute \( M_{ux} = M_{wy} = 1.0 \) kN.m/m, \( L_x = 9.4 \) m and \( L_y = 2.4 \) m. With this substitution the two results are found to be within 2.9%, which is sufficient accuracy for the purpose of this research and indicates the 45° assumption is valid. Zhang [1998] showed that if the wall has fixed supports at the base and two sides, the difference between the true solution and the 45° solution is 1.5%, and Park’s assumption is even more accurate.

5.3.4 Test Specimen with Windows

The analysis of the test specimen can be completed in the same manner. The ultimate moment capacity of the vertically spanning 800mm wide strip was found in Table 5.1 to be 3.2 kN.m. Dividing by 0.8 converts this to the moment capacity per unit width of \( M_{sy} = 4.0 \) kN.m/m.

To determine the internal work about the y-axis we must consider the post-elastic moment capacities of the bond beam and horizontally spanning masonry strips. As unreinforced masonry has a very low tensile capacity, and there is no horizontal steel in the lower courses of the wall, we can assume that in the post-elastic region where yield line analysis is applicable the lower masonry courses do not contribute to the moment capacity of the wall. This means that if a yield line rotates about a vertical axis, the only internal work done will be the bond beam capacity multiplied by the rotation. In this situation to simplify notation the bond beam capacity has been smeared over the height of the wall, so \( M_{ux} = \) bond beam capacity / height of wall \((L_y) = 9.9 \) kN.m / 2.4 m = 4.1 kN.m/m. This smearing of strength does not affect the result of the analysis.
The yield line analysis was completed using 45° as the yield line angle. If the yield lines were found to deviate considerably from this, then the analysis would have been repeated using the actual crack pattern. Figure 5.8 shows the assumed yield line pattern on the test specimen.

![Diagram showing yield line pattern](image)

Figure 5.8 – Proposed yield line pattern for test specimen

The internal work calculations for this wall are as follows:

**Table 5.6 – Internal work for test specimen**

<table>
<thead>
<tr>
<th>Panel</th>
<th>$\partial_x$</th>
<th>$\partial_y$</th>
<th>$M_{ux}y_0\partial_x$</th>
<th>$M_{uy}x_0\partial_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABC</td>
<td>$\Delta/L_y$</td>
<td>0</td>
<td>$2M_{ux}L_y\Delta/L_y$</td>
<td>0</td>
</tr>
<tr>
<td>ABDF</td>
<td>0</td>
<td>$\Delta/L_y$</td>
<td>0</td>
<td>$M_{uy}L_x\Delta/L_y+(2L_y-L_w)M_{uy}/L_y$</td>
</tr>
<tr>
<td>DEF</td>
<td>$\Delta/L_y$</td>
<td>0</td>
<td>$2M_{ux}L_y\Delta/L_y$</td>
<td>0</td>
</tr>
</tbody>
</table>

The external work is calculated by determining what the total work would be on each panel if there was no opening and subtracting the work done on the opening. This is much easier than breaking each panel down into simple shapes and evaluating each simple shape’s area and centroid location.
Table 5.7 – External work on test specimen

<table>
<thead>
<tr>
<th>Panel</th>
<th>External Work</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABC</td>
<td>( \frac{1}{3} w_u L_x L_y \Delta - \frac{1}{2} w_u L_w L_y \frac{L_w}{L_y} \Delta )</td>
</tr>
<tr>
<td>ABDF</td>
<td>( \frac{1}{2} w_u (L_x - 2L_y) L_y \Delta - \frac{2}{3} w_u L_w L_y \frac{L_w}{L_y} \Delta )</td>
</tr>
<tr>
<td>DEF</td>
<td>( \frac{1}{3} w_u L_x L_y \Delta - \frac{2}{3} w_u L_w L_y \frac{L_w}{L_y} \Delta )</td>
</tr>
</tbody>
</table>

The external and internal works are summed across all panels and equated. The resulting expression is solved for \( w_u \) and the following equation obtained:

\[

w_u = \frac{4M_{ux} + 2M_{wy} + M_{wy} \frac{L_x}{L_y} - M_{wy} \frac{L_w}{L_y}}{\frac{1}{2} L_x L_y - \frac{1}{3} L_y L_x - \frac{11}{6} L_w L_y}

\]

The values obtained at the start of this section and the geometry of the wall give the following values to be substituted into this equation.

Table 5.8 – Test specimen yield line analysis values

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{ux} )</td>
<td>4.1</td>
<td>kN.m/m</td>
</tr>
<tr>
<td>( M_{wy} )</td>
<td>4.0</td>
<td>kN.m/m</td>
</tr>
<tr>
<td>( L_x )</td>
<td>9.4</td>
<td>m</td>
</tr>
<tr>
<td>( L_y )</td>
<td>2.4</td>
<td>m</td>
</tr>
<tr>
<td>( L_w )</td>
<td>1.0</td>
<td>m</td>
</tr>
</tbody>
</table>

Once these values are substituted we obtain a value for the ultimate load capacity of the wall of 4.0kPa.
5.3.5 Conclusions

An opening or openings in a wall can affect its ultimate load carrying capacity in two ways. An opening is an area of zero moment carrying capacity. This will reduce the internal work done, and consequently reduce the total load capacity. An opening is also an area where the external load on a wall is not applied. This will reduce the external work done, and tend to increase the ultimate load capacity. Clearly the number, size and location of openings could have a widely varied effect on a wall.

Zhang [1998] completed yield line analysis on several specimens to evaluate the effect of openings in different locations. He found that if a wall similar to the test specimen had a door opening at each end, the reduction in load capacity would be 47% relative to the wall with no openings. This is consistent with what would be expected – the yield lines have been considerable shortened, and the loading area affected little. However, if a test specimen had a large window in the centre of the wall, where it is inside both yield lines, then it reduces the load on the wall without reducing it's capacity. In this situation, the wall was found to be able to carry 16% more load that the wall with no openings. Figure 5.9 illustrates these two cases.

![Capacity 53% of solid wall](image1)

![Capacity 116% of solid wall](image2)

Figure 5.9 – Variation in yield load for different opening locations

As yield line analysis is an upper bound method, if the wrong crack pattern is selected it will prove to be non-conservative. For simple analyses this is not a problem as the optimum yield line pattern can be solved for analytically. For more complicated analyses, however, a designer may fail to consider the critical crack pattern which gives the lowest, and most accurate, load capacity of a wall or slab. A technique which automatically identifies the correct yield line pattern is desirable; one method that has been developed is described below.
5.4 Combined Analysis

Seah, Dawe and Dukuze [1993] developed a computerised approach to the analysis of masonry walls with complicated geometry and unknown yield line locations. The program develops a finite element model of the wall and uses this model to evaluate elastic bending moments and from this determine likely locations for initial cracking. Figure 5.10 gives an example of this.

These cracking locations and yield line geometry are transferred to a yield line analysis, which solves for the predicted ultimate load for that yield line pattern. The pattern is then varied iteratively to maximise external work and minimise internal work. This combination gives the lowest ultimate load capacity, which will be the most accurate answer. A sample solution has been shown in Figure 5.11.

This system was used to analyse 90 masonry walls intended for a nuclear power plant, some of which had complicated openings and boundary conditions. The final solution for these complicated panels was often quite different from that a designer might estimate to be correct, hence this system provides a good check, as the lowest solution in yield line analysis is closest to the true solution. Graphical output allowed easy checking of wall opening location and boundary condition input, and verification of the final yield line pattern. The calculated strengths of the panels were found to be substantially higher than that required by the current codes and standards.

![Diagram](image)

Figure 5.10 – Moment contours and initial yield lines for wall with door opening
This computerised combination of finite element analysis and yield line analysis must be considered the most foolproof and easiest method for determining ultimate load of complicated slabs and walls with many geometric variables and complicated boundary conditions.

5.5 Force-Deflection Yield Line Analysis

Finite element analysis provides a good model of load-deflection to yield, but gives inaccurate information beyond. Traditional yield-line analysis provides a failure load but does not give any indication of the deflection associated with this load. A model representing the deflection of the wall past yield is required to fully explain the behaviour of these walls.

It has been noted from previous research that in the post-elastic region, almost all deflection is associated with the opening of large cracks at the base, edge and other yield lines of the wall. The considerable width of these cracks can be attributed to the steel strain hardening, increasing in strength and causing bond-slip to occur, invalidating the usual assumption that plane sections remain plane.

Assuming a simplistic model of bond slip with yield penetration length of 10d_b and a linear variation of strain at the bar's ultimate strength, as shown in Figure 5.12, the elongation of the bar can be predicted. This elongation can be converted to a rotation about the centre of the compressive block. This is a very crude model of moment-
rotation. Experimental results would give a better model that could easily be incorporated into this analysis. Figure 5.13 and Figure 5.14 show the stress-rotation relationships for the wall base and the bond beam.

Figure 5.12 – Assumed strain profile in rebar as bond slip occurs

Figure 5.13 – Stress-rotation relationship for the wall base
For a given value of steel stress, the moment across the masonry crack can be calculated, as the required compressive force, hence the stress block size, hence internal moment arm may be found. Figure 5.15 and Figure 5.16 show the result of this calculation. As we are trying to obtain the post-elastic behaviour of the wall, the point on the finite element analysis where bar yield occurs will be used as a datum for load and deflection. To this end, the moment calculated across the crack is reduced by the value of the yield moment, and we will use the remaining “post-yield moment” in this analysis. Deflections obtained will be added to the yield deflection to obtain total deflection.
Figure 5.16 – Bar stress / joint moment relationship for the bond beam

A relationship for moment-rotation in the post-elastic region for a crack can now be found using the steel stress as a driving variable. These were calculated for the 800mm vertical strip of wall and the bond beam for use later in the analysis. Figure 5.17 and Figure 5.18 display these relationships.

Figure 5.17 – Rotation-moment relationship for the bond beam
Figure 5.18 – Moment / rotation relationship for the wall base

A model of the specimen is next constructed. The centre of the model has a deflection imposed upon it, and this is used to obtain the rotation about the support lines. This rotation is then used to calculate the moments applied to each panel by internal bending action across the cracks. This will be identified as $M_{ci}$, where $i$ is the panel identifier. Figure 5.19 shows the actions of these moments.

Figure 5.19 – Rotation of panels and action of support moments

As is readily apparent, when using the 45° yield line for the test specimen, the rotation for all panels is $\Delta/L_y$. 
The shears at the intersections of the panels will be distributed along the length of the panel in an indeterminate manner. The exact distribution is not required once it is realised that over any short length, the shearing force will have the same moment-arm about the supports of both panels. Thus the shears can be represented as an identical moment, $M_{y}$, acting about the base of each panel, where $j$ is the internal yield line identifier. In the large panel, this “shear” moment will be in the same direction as the internal moment action, as the side panels are supporting it. Conversely, in the side panels, the “shear” moment will be opposing the internal moment action, as these panels are supporting the inner panel. Figure 5.20 displays the action of these “shear” moments.

![Diagram](image)

Figure 5.20 – $M_{y}$ with 45° yield line

This does not require the yield lines to be at 45°, and the factor on $M_{y}$ for two adjoining panels to be equal. For example, if the yield lines were at 26.6° to the horizontal, $M_{y}$ for each yield line in the base panel would be $\frac{1}{2}$ of $M_{y}$ in the edge panels, as at each small increment of shear, the moment-arm is half as long [$\tan (26.6°) = 0.5$].

The moment caused by the applied load about the panel support is now calculated from the area of the panel and the distance from the panel centroid to the support. The simplest
method is to take the entire panel as one entity and remove any holes afterward. This moment is represented as $M_{wi}$, with $i$ being the panel identifier. This moment has been evaluated for each panel as follows.

Table 5.9 – Applied load moment on each panel

<table>
<thead>
<tr>
<th>Panel</th>
<th>Applied load moment, $M_{wi}$ (Algebraic)</th>
<th>Applied load moment, $M_{wi}$ (Geometric values substituted)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$w_{uo}\left(\frac{1}{6}L_\pi L_\rho L_\gamma - \frac{1}{2}L_u L_w L_w\right)$</td>
<td>1.804m$^3$ x $w_{uo}$</td>
</tr>
<tr>
<td>2</td>
<td>$w_{uo}\left(\frac{1}{3}L_\pi L_\rho L_\gamma + \frac{1}{2}(L_x - 2L_y)L_\gamma L_\rho - \frac{2}{3}L_u L_w L_w\right)$</td>
<td>17.19m$^3$ x $w_{uo}$</td>
</tr>
<tr>
<td>3</td>
<td>$w_{uo}\left(\frac{1}{6}L_\pi L_\rho L_\gamma - \frac{2}{3}L_u L_w L_w\right)$</td>
<td>1.637m$^3$ x $w_{uo}$</td>
</tr>
</tbody>
</table>

It must be noted that $w_{uo}$ in this analysis represents the extra load over and above the yield load, as it is this extra load that causes post-elastic deformation. For the specimen tested, the components of this moment equilibrium of each panel have been evaluated. Moments resisting rotation are added, moments causing rotation are subtracted. These moments have been displayed algebraically in Table 5.10 and graphically in Figure 5.21.

Table 5.10 – Moment equilibrium components for each panel

<table>
<thead>
<tr>
<th>Panel</th>
<th>Moment equilibrium components</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$M_{c1} - M_{s1} - M_{wi} = 0$</td>
</tr>
<tr>
<td>2</td>
<td>$M_{c2} + M_{s1} + M_{s2} - M_{w2} = 0$</td>
</tr>
<tr>
<td>3</td>
<td>$M_{c3} - M_{s2} - M_{w3} = 0$</td>
</tr>
</tbody>
</table>
Figure 5.21 – Moment equilibrium components for each panel

This system of equations can be reduced by cancelling “shear” moments $M_{s1}$ and $M_{s2}$ to

$$M_{c1} + M_{c2} + M_{c3} - M_{w1} - M_{w2} - M_{w3} = 0$$

Substituting the values from Table 5.10 for $M_{w1}$ into this equation we get

$$M_{c1} + M_{c2} + M_{c3} = 20.63 \, w_{uo}$$

For a given deflection the moments $M_{c1}$, $M_{c2}$ and $M_{c3}$ can be found from the geometric considerations and moment-rotation relationship developed earlier. Once these values are substituted into the above equation the face load for equilibrium may be evaluated for the nominated deflection. This method was solved at a number of deflections and the loads evaluated. The predicted behaviour has been included for comparison with the experimental data. Figure 5.22 gives the additional deflection due to bond slip and additional load due to strain hardening. Figure 5.23 uses this data with that of the ABAQUS analysis, with the final point of the finite element analysis being used as the starting point of the load-deflection yield line analysis. This was deemed appropriate because the ABAQUS analysis did not take into account the effect of strain hardening, and terminated when bond slip began to occur.
Figure 5.22 – Additional Deflection and Load due to Bond Slip and Strain Hardening

Figure 5.23 – Force-deflection yield line analysis response

The experimental and theoretical applied loads remain within 8% up to 75mm deflection. Above 75mm deflection the increase in experimental load recorded causes considerable differences between theory and experimental results. This has been attributed in part to the recorded pressure being too high due to airbag bulging and loss of contact area, and in
part to an increase in wall capacity due to the catenary effect as described in Section 7.2.5.

Several assumptions were made in the development of this analysis that will affect its accuracy. The yield lines will not all begin strain hardening at the same wall deflection, as has been assumed. With a more gradual onset of strain hardening the increase in load will be spread out over a greater wall deflection, making for a less sharp increase in load just past the point of first strain hardening. This agrees with what is seen in the theory/experimental comparison in Figure 5.23 above, where the sudden increase in load resisting capacity due to strain hardening was inconsistent with experimental results.

The model of moment-rotation is a very simple one based on assumed bond slip parameters. If a better model was developed through experimental testing or improved analytical work, the accuracy of the analysis should improve.
6 Test results and Observations

6.1 Testing Overview

The test specimen was built in the Structural Engineering Laboratory of the University of Canterbury. The location in the laboratory was chosen to shield the wall from the sun as much as possible to prevent strain gauge drift. 61 strain gauges set up in ¼ bridge configuration recorded rebar strain at critical locations, and 13 potentiometers recorded displacements at the top of the main and return walls. 104 DEMEC points were installed to measure strain at the top of both faces of the main and return walls. In addition, DEMEC points were installed to allow the in-plane bending and shear of the return walls to be evaluated.

Testing took place over the week 29/01/01 to 02/02/01. Testing took a week because for each half-cycle the three reaction frames had to be disconnected and shifted to the opposite side of the wall, and the air supply system unattached and reattached.

6.2 Loading Regime

Serviceability Limit State (SLS) and Ultimate Limit State (ULS) loads calculated from NZS 4203:1992 for this type of wall were evaluated by Zhang [1998] as 0.26kPa and 1.6kPa respectively. This assumes the wall is in a high seismic risk zone, appropriate code factors, and that the wall responds elastically. It also assumes a fundamental period of $T=0.65\text{s}$ (from $T=0.09HL^{0.5}$ [Paulay, 1992]) and a wall mass of 290kg/m$^3$. The ULS pressure is equivalent to a self-weight seismic acceleration of 0.55g. The SLS and ULS loads were included in the loading regime as benchmarks to allow comparison between code expectations and actual response.

Load cycles before yield were load controlled, and cycles beyond yield were displacement controlled. Yield was defined by Zhang et al [2001] to have occurred when all of the starter bars along the base of the wall had yielded. However, due to strain instrumentation difficulties, the yield displacement was determined at the end of testing.
from the intersection of the initial and final stiffnesses, and the yield load from the test pressure at this displacement.

Positive loading is defined as pushing from the side with the return walls; negative loading is defined as pushing towards the return walls. Figure 6.1 shows the loading pressure at each cycle, and Figure 6.2 shows the midspan bond-beam displacement at each cycle.

![Figure 6.1 - Loading regime (pressure)](image1)

![Figure 6.2 - Loading regime (central displacement)](image2)
6.3 General Behaviour Observed

At the start of the test there were a few observable shrinkage cracks above both the corner and face window; these cracks were all <0.1mm wide. The first cracks due to loading occurred at +2.2kPa and −1.6kPa. At low levels of load the only crack of substantial length was at the base of the wall. All cracks closed fully when load was released until a load level of 4.8kPa was applied, which left some very fine cracking visible after the load was released. The hysteretic behaviour for these pre-yield cycles has been plotted as Figure 6.3.

![Graph showing Load (kPa) vs. Midspan Deflection (mm)](image)

Figure 6.3 – Hysteretic behaviour before yield

The first crack above the window in the face of the wall occurred at a load of +2.2kPa. At a load of +4.8kPa a crack formed above the corner window, and there were several cracks in the bond beam. These cracks continued to widen and other cracks formed in nearby locations as cyclic loading continued.

Return wall cracking was first observed at +4.8kPa and −4.6kPa. These cracks were found to be consistent with the in-plane, out-of-plane and torsion loads applied to the return walls.

Hysteretic behaviour to +/- 25mm midspan deflection has been plotted as Figure 6.4. Figure 6.5 shows all cycles of loading applied to the wall, and Figure 6.6 shows an envelope of hysteretic behaviour including the failure cycle.
Figure 6.4 – Hysteretic behaviour out to +/- 25mm midspan deflection

Figure 6.5 – Hysteretic behaviour for all cycles
Yield was determined to have occurred at a load of 3.6kPa and a deflection of 8.54mm. This was evaluated from the intersection of the initial and post-yield tangent stiﬀnesses. This graphical calculation has been plotted as Figure 6.7.

Maximum crack widths at 1% drift were 1.75mm at the base of the wall and 0.4mm in the bond beam. Hairline cracking of blocks was observed at this load level. At 2% drift the maximum base crack width was 3.5mm, and the widest crack in the wall was 1.5mm wide, measured in the bond beam above the corner window. Cracks in blocks began to open up, with the widest measured at 1.2mm.

The width of cracks observed was considerably less than that observed in previous research [Singh, Zhang]. This was attributed to the more widespread cracking, with wall rotation not being concentrated across one crack. The progression of cracking can be seen in Appendix 3.

The blocks above the corner window performed excellently, showing less cracking and smaller crack widths than observed on Zhang’s specimen that did not use the clip-bar detail. This confirms that this reinforcing detail provides enhanced moment capacity around bond-beam corners.
At the point of first yield it was observed that the wall’s stiffness began to degrade. This degradation became significant during the +1% loading cycle and continued through further loading cycles, with the wall having very little stiffness when unloaded, resulting in the pinched nature of the post-elastic hysteresis curve. For the load cycles of 1% inter-storey drift (24mm) and beyond it was found that the wall displayed similar stiffness’s on the unloading-reloading-unloading part of each cycle. This can be seen in Figure 6.8, a plot of the +2% cycle only.

The load resisting capacity of the wall continued to climb with each load cycle. The load at yield was 3.63kPa, but at 200mm deflection on the failure cycle the wall had reached 8.2kPa. Past 200mm the recorded load continued to climb, but as the airbags were reaching the limit of their travel these figures do not accurately represent the load applied to the full face of the wall. The first catastrophic damage occurred in the bond beam at the intersection of the main wall and return wall B at a midspan deflection of 231mm, which represents an inter-storey drift ratio of 9.6%.

![Graph](image.png)

Figure 6.7 – Determination of yield load and displacement from hysteretic envelope
6.4 Experimental Results

A total of 20 loading cycles were applied to the test specimen. Of these 6 were in the elastic range and 14 were applied post-yield. This section presents deflections, strain distributions and observations of cracking at each load level.

Positive 0.26kPa (SLS loading)

At the code-level serviceability limit state load, no strain was recorded. No cracking occurred and the only recorded deflection was 0.8mm at the centre of the bond beam.

Negative 0.26kPa (SLS loading)

In the reverse direction the serviceability loading again failed to register strain or cause cracks. The bond beam again deflected 0.8mm at midspan.

Positive 1.6kPa (ULS loading)

The code-level ultimate limit state load failed to cause cracking in the positive direction. The recorded bond beam central deflection was 1.7mm. Figure 6.9 and Figure 6.10 show pre-yield displacements along the centre of the bond beam.
Figure 6.9 – Pre-yield bond beam displacements due to positive loading

Figure 6.10 – Pre-yield bond beam displacements due to negative loading

Return wall bond beam displacements before yield are shown in Figure 6.11 and Figure 6.12. At this level of loading, strains were first registered on the strain gauges, with a
maximum strain of 132 microstrain being observed. The starter bar strain distribution for positive loading before yield has been included as Figure 6.13, and those for negative loading as Figure 6.14. At this loading level and for all cycles before yield the bond beam showed no measurable strain.

Figure 6.11 – Return wall A bond beam displacements before yield

Figure 6.12 – Return wall B bond beam displacements before yield
Negative 1.6kPa (ULS loading)

The ultimate limit state load caused a crack across the base of the wall when applied in the negative direction. This crack in the mortar was too fine to be measured and closed up completely when the wall was unloaded. A maximum starter bar strain of 203 microstrain was recorded, and the maximum observed deflection was 2.2mm at the centre of the bond beam.
Positive 2.2kPa (full base crack)

The specimen was loaded in the positive direction until a base crack formed and extended fully along the length of the wall. This required a load of 2.2kPa and caused a deflection of 3.3mm at the centre of the bond beam. The maximum strain recorded in a starter bar was 324 microstrain. At this loading level the return walls showed increasing rotation, but little change in overall deformation, relative to the +1.6kPa cycle. A crack in the bond beam above the face window was noted. All cracks were very fine and closed up completely upon removal of the load. A residual deflection of 0.6mm was recorded.

Deflection profiles and strain gauge readings suggest that the return walls were beginning to provide some restraint to the main wall. The wall’s behaviour was shifting from that of an unpropped cantilever to that of a propped cantilever.

Negative 1.9kPa (full base crack)

The load required to cause a crack fully across the base was slightly lower in the negative direction. A similar deflection at the centre of the bond beam of 3.25mm was recorded. The crack at the base of the wall was too fine to measure, and were invisible when the wall was unloaded. There were no other cracks recorded at any other location. Strain readings in all starter bars increased relative to the previous negative cycle, to a maximum of 324 microstrain. A residual deformation of 0.3mm was recorded after load was removed.

Both return walls were pushed outward 0.8mm at the free ends under this loading cycle. It was observed that the return walls showed increased rotation but no overall deflection relative to the −1.6kPa loading cycle.

Positive 4.8kPa (first recorded starter bar yield)

The application of load in this cycle was controlled by the strain in the starter bars at the base of the wall. Load was applied until the first bar reached a strain of 1500 microstrain, the yield strain. Upon examination of the load-deflection curve after the load had been applied it was determined that structural yielding had occurred at a deflection less than
the wall had undergone. This possibly may be because the starter bars were yielding at locations slightly above or below the strain gauges, giving lower levels of recorded strain than the physical maximum and causing overestimation of the deflection required to yield a starter bar. The strain gauges were installed on the starter bars at the level of the mortar bedding joint.

The first recorded starter bar yield occurred in the central starter bar at a bond beam midspan displacement of 17.5mm. Bond-beam deflections due to positive loading from this point onwards have been plotted as Figure 6.15 and Figure 6.16. It can be seen that the bond beam is forming a rounded shape that is not what would usually be associated with the failure pattern suggested by yield line analysis, nor with the test done by Singh [1998] and Zhang [1998]. This rounded shape continued through all following cycles.

The free ends of the return walls were pulled in 1.9mm (A) and 1.8mm (B) in this loading cycle. The return wall deflections under positive loading from this point on have been included as Figure 6.17 and Figure 6.18. The strain in the starter bars varies considerably along the base of the wall – while the central bar is registering 1500 microstrain, the yield strain, the two starter bars 900mm and 700mm in from the return walls show lower strains of 326 and 842 microstrain. This disparity suggests that the wall is not yet behaving as yield line theory suggests it should as it begins to fail. Starter bar strain gauge readings for this and all following cycles has been plotted as Figure 6.19 and Figure 6.20 for the main wall, and Figure 6.21 and Figure 6.22 for return walls A and B. The strains in the bond beam steel increased considerably in this cycle, with a maximum value of 547 microstrain being recorded at the intersection of the corner window and the wall. Bond beam strains post-yield have been plotted as Figure 6.23 and Figure 6.24.

A considerable increase in cracking was observed during this cycle. This occurred at the bases of the return walls, in the bond beam at the corners of the windows and in the face of the wall as the beginnings of the diagonal yield-line pattern. The crack at the base of the wall was 0.8mm wide, and the widest crack in the bond beam was 0.15mm, above the corner window where the main and return walls meet. All other cracks were too fine to measure. When the wall was unloaded, for the first time cracks were still visible but too fine to measure at the base of the wall and in the bond beam above the corner window.
Figure 6.15 – Post-yield bond beam displacements due to positive loading

Figure 6.16 – Post-yield bond beam displacements due to negative loading
Figure 6.17 – Return wall A post-yield displacements

Figure 6.18 – Return wall B post-yield displacements
Figure 6.19 – Post-yield starter bar strain gauge readings under positive loading

Figure 6.20 – Post-yield starter bar strain gauge readings under negative loading
Figure 6.21 – Return wall A post-yield starter bar yield strains

Figure 6.22 – Return wall B post-yield starter bar yield strains
Figure 6.23 – Post-yield bond beam strains due to positive loading

Figure 6.24 – Post-yield bond beam strains due to negative loading
The vertical cracking on the inside face, where the main wall meets both return walls suggests that there is significant moment being transferred across these interfaces, and that the return walls are providing considerable restraint to the main wall. The rotation of these interfaces has been plotted against the applied load as Figure 6.25.

![Graph showing load vs. rotation for return wall and main wall intersections.]

Figure 6.25 – Return wall / main wall interface rotations

Negative 3.0kPa

As the difference in displacement between the two previous cycles in the positive direction was so great, it was decided to load in the negative direction to a smaller load to study the behaviour between cracking and yielding more closely.

Under a load of 3.0kPa the wall displaced 9.75mm at the centre of the bond beam. The residual deflection at this point when unloaded was 0.9mm. The maximum deflection of both return walls was 2.1mm outwards at the free ends. A maximum strain of 659 microstrain was recorded in the central starter bar, and the maximum strain in the bond beam was 317 microstrain. No residual strain had been recorded in the central starter bar after the previous cycle, as it was taken to yield and no further, with the system remaining in its elastic displacement range.
The crack along the base of the wall increased in length slightly under this load, with a maximum recorded width of 0.75mm. New cracking was observed in the bond beam above both windows, and a diagonal crack formed through the diagonally opposite corners of the face window that exactly matches yield line theory predictions. The first crack through a block was observed in the bond beam above the corner window in the return wall.

Negative 3.0kPa (repeat loading)

It was possible that a slight yielding of the wall in the positive direction had compromised the elastic response in the negative direction. To investigate this possibility, the load of 3.0kPa was repeated in the negative direction. This cycle proved virtually identical to the last, with no new cracking and almost identical deflections and strains recorded everywhere. It was concluded that the slight positive yield had not affected the negative elastic response.

Negative 17.5mm

The wall was next loaded in the negative direction until a starter bar strain gauge registered yield. Again this occurred at 17.5mm deflection, but at the slightly lower load of 4.7kPa. A considerable increase of cracking was observed, with many fine cracks developing in the face of the wall near the corner window, many of which crossed the bond beam at some point. There was an increase in bond beam cracking above the windows, and a number of vertical mortar cracks in the outside faces of return wall B and the main wall where the two intersected. These cracks are consistent with the transfer of moment around this corner. The crack locations in the bond beam above the corner window indicate that moment is being transferred around this corner to the bond beam.

When the load was removed fine residual cracking was visible along the base of the wall, and along the diagonal yield line running through the corner window. All other cracks closed so as to be undetectable.
The tensile strains in the starter bars of the return walls were increasing as load increased. It was noted that the maximum return wall starter bar strain of 491 microstrain was greater than the maximum return wall starter bar strain of 460 microstrain at 17.5mm deflection in the positive direction. It was also noted that the out-of-plane deflections of the free end of the return walls were nearly twice as high for negative loading as positive loading. This is attributed to the different moment capacity and stiffness of the bond beam corner detail under opening and closing. This phenomena was tracked throughout testing and found to become more prominent as deflection increased – at +4% drift the maximum return wall deflection was 4.1mm, and at −4% drift this was 23.7mm, 5.7 times as great. The strains in the starter bars for negative loading past wall yield were 2.5 to 3 times as high compared to similar positive cycles. This leads to the conclusion that the strain in the return wall steel is predominantly affected by out-of-plane actions, with in-plane actions having less influence.

This conclusion is supported by the return wall starter bar strain gauges, which always indicated tension. Under the in-plane loading on the return walls compression would be expected in some of the starter bars. However, the tension in all bars due to out-of-plane loading was great enough to cancel out this compression strain.

Positive 17.5mm (repeat loading)

The previous maximum positive deflection was repeated to evaluate if the stiffness in the positive deflection had been reduced by the three negative loading cycles. When this deflection was reached the applied load was 3.7kPa, down 23% from the previous load at this deflection of 4.8kPa. However, the hysteresis curve shows that the tangent stiffness was the same at this deflection for both cycles, with the decrease in load being caused by an initial lack of stiffness due to the wall rocking slightly. This slight rocking suggests that the bars have yielded slightly.

Almost identical strain and deflection data was recorded for the two cycles at this deflection. There was an increase in cracking that consisted of the extension of previously existing cracks. The base crack was measured at 0.75mm, with 0.1mm of this remaining open once unloaded. A 0.3mm crack was recorded in the corner of the bond beam above the corner window, which was visible but too fine to measure once load was
removed. All cracking in the face of the wall away from the yield lines was too fine to measure and closed up completely when unloaded. The residual deflection 4.0mm was the same for both cycles to this deflection.

Positive 1% drift

For the displacement controlled cycles from this point onwards, the degradation of load carrying capacity under repeated loading was recorded. The wall was loaded to the target displacement, in this case 24mm, and immediately unloaded. The airbags were disconnected to allow any residual pressure out of the system, then they were reconnected and the wall again pushed to the target displacement. This meant that the first loading would be taking up the residual drift from the previous load cycle, and the second loading would have no residual drift to take up.

At an interstorey drift level of 1%, or 24mm displacement at the centre of the bond beam, the pressure when first loaded was 4.8kPa. When the load was removed a residual deformation of 5.3mm was observed, and when the wall was reloaded the pressure was 4.4kPa. When this was released, the residual deformation was 6.5mm.

The free ends of both return walls deflected outwards 2.2mm. The maximum starter bar strain was 1814 microstrain in the central starter bar and the maximum strain in the bond beam was 634 microstrain above the corner window. The maximum strain in a return wall starter bar was 644 microstrain in the bar at the intersection of the main wall and return wall A.

The base crack opened up to 1.4mm on the tension side, and for the first time the base crack on the compression face was still visible, indicating the starter bars had been yielded and that plastic strain had occurred. The largest crack above the corner window was 0.4mm wide. The cracks above the face window were all less than 0.1mm wide. An increase of the number of fine diagonal cracks on the wall face was also observed, but these were all very fine and could not be measured.

Cracking was observed on both sides of both return walls. Cracking on the outside faces at the free end base is consistent with in-plane and out-of plane bending of the return.
walls. An angled crack pattern on the inside of return wall B is what would be expected due to the torsion about a vertical axis being applied to the return wall.

It was observed that the unloading paths for the first and second application of load were very similar, a pattern which continued throughout the following cycles.

Negative 1% drift

The pressure when first loaded was 4.6kPa. When the load was removed a residual deformation of 3.7mm remained, with the reload pressure being 4.4kPa. After load release, a residual deformation of 5.0mm remained. This indicates that plastic deformation and some steel relaxation occurred when the load was held for approximately ½ an hour on the reload to allow DEMEC reading and crack marking.

Return wall deflections of 4.9mm (A) and 5.2mm (B) were recorded. Return wall starter bar strains had a maximum value of 614 microstrain, which is comparable to that of the the positive cycle. The maximum strain in a main wall starter bar was 1520 microstrain, just over the yield strain. The maximum strain in bond beam steel was 409 microstrain, recorded in the corner steel 100mm in from return wall A.

This cycle saw an increase of cracking above the face window, with the widest crack being measured at 0.1mm. The diagonal yield line crack at the bottom corner of this window was 0.2mm wide. An increase in cracking above the corner window was observed, with the widest crack being 0.3mm. The crack along the intersection of the main wall and return wall B opened to 0.25mm and lengthened slightly, and broke across two blocks. Fine diagonal torsion cracks began to form in return wall B. The base crack opened to 1.75mm, and remained 0.35mm wide when unloaded. All other cracks were too fine to measure when the load was removed.

Positive 2% drift

At 48mm displacement at the centre of the bond beam, the pressure when first loaded was 5.8kPa. A residual deformation of 17.2mm was observed, and when the wall was reloaded the pressure was 5.2kPa. A final residual deformation of 19.1mm was observed.
The crack along the intersection of the main wall and return wall B opened to 1.1mm during this cycle, and remained open 0.5mm after unloading. The widest crack above the corner window was 1.2mm wide, closing to 0.35mm. The widest crack above the face window was 0.3mm wide, closing to 0.1mm. The yield line through the face window opened to 1.2mm, of which 0.7mm was recovered after unloading. The base crack was measured at 3.1mm, closing to 1.3mm.

An extension of the diagonal cracks on both sides of return wall B was observed, with some vertical block splitting on the outside face. There was considerable fine cracking in the wall face away from the yield lines, which were still visible but too fine for measurement when the load was removed.

During this load cycle starter bar 13 (on the return wall B side of the face window) yielded for the first time. This had the maximum recorded strain of 1634 microstrain. The maximum starter bar strain in a return wall was 755 microstrain at the free end of return wall A. Return wall A pulled inward 4.0mm and return wall B moved inwards 3.7mm. The bond beam displacement profile shows a smooth variation of displacement along the face of the main wall.

It was noted that the clip bar strain gauges were showing compression. Tension would be expected given that the clip bars are on the inside of a joint that is opening. The strain gauges were located directly under a large crack, where the bond beam rotation concentrated. This concentrated rotation would have caused the clip bars to bend. Because of the positioning of the strain gauges on the cross-section of the clip bars, they registered the compression due to bending. This effect was noticed in the other loading direction in later cycles, where the bar should have been in compression but bending was causing tension strains to register on the instrumentation.

Negative 2% drift.

At an interstorey drift level of −2% the pressure when first loaded was 5.7kPa. When the load was removed a residual deformation of 12.4mm was observed, and when the wall
was reloaded the pressure was 5.1kPa. When this was released, the residual deformation was 14.5mm.

The maximum strains recorded in the main wall and return wall starter bars were 1338 and 1309 microstrain respectively. It was noted that the maximum starter bar strain was below the yield strain, while for the −1% cycle it was 1520 microstrain, above yield. This has been attributed to increased cracking which forces a more even distribution of starter bar strains. The maximum strain in the bond beam was 983 microstrain in the outside corner bar at the intersection with return wall B. Maximum return wall deflections of 11.7mm (A) and 9.7mm (B) were recorded.

The new cracking in the face of the wall was mostly the extension of and connection between cracks from previous negative loading cycles. The diagonal crack on the outside of return wall B reached both sides of the wall. On the outside face of return wall A a horizontal crack formed in the mortar joint immediately below the bond beam. This horizontal crack could indicate one of two things – firstly, that a sliding failure might be possible in return wall A because of the decreased return wall length, and secondly, that torsion in the bond beam is providing restraint to the return wall and forcing it into double curvature. As no crack was observed on the inside of the wall the second option is considered most likely.

Cracks from previous positive load cycles were visible during this cycle. These cracks had previously closed under negative load so as to be invisible, but due to increasing crack widths and wall damage this was no longer possible.

A strain gauge installed on the vertical steel at the location of the yield line crack running through the bottom of the face window indicated the reinforcing at this location was yielding, with a strain of 1674 microstrain. This was the first recorded yielding of vertical steel.
Positive 3% drift

At an interstorey drift level of 3% the pressure when first loaded was 6.1kPa. A residual deformation of 27.3mm was observed, and when the wall was reloaded the pressure was 5.5kPa. When this was released, the residual deformation was 27.8mm.

Three starter bar strain gauges now recorded strains in excess of the yield strain, with the maximum being 2668 microstrain. The maximum return wall starter bar strain was 890 microstrain. The maximum return wall deflections were 4.2mm (A) and 4.6mm (B).

Little additional cracking was observed, with the existing cracks opening wider to take up the wall deformation. The yield line crack at the bottom corner of the face window opened to 3mm under this loading cycle, with a 2.5mm out-of-plane discontinuity across this crack. A large increase in crack size was expected, as it was this location at which the vertical steel yielded in the −2% drift cycle. The face and corner windows had 0.75mm and 1.5mm cracks above them, respectively. There was an increase in diagonal cracking in both return walls. The base crack was 4.5mm wide, dropping to 0.8mm when unloaded. Cracks at the intersections of the main and return walls remained 1.0mm wide after unloading.

Negative 3% drift

6.1kPa was required to cause -72mm displacement at the centre of the bond beam. When the load was removed a residual deformation of 18.2mm was observed, and when the wall was reloaded the pressure was 5.5kPa. When this was released, the residual deformation was 20.3mm.

The trend of return wall deflections being much higher for negative load cycles continued, with the free end deflections being 19.0mm (A) and 16.4mm (B) outwards. The maximum starter bar strain in the main wall was 2374 microstrain. For the first time, yielding of the return wall starter bars was observed, with 1698 microstrain being recorded at the free end of return wall A. The strain at the free end of return wall B was 1368 microstrain. The first yielding of the corner detail steel occurred, with the clip bar at the return wall B end registering 1540 microstrain. The maximum strain along the
bond beam was 694 microstrain, recorded where the yield line crack crossed the bond beam above the face window.

At this deflection level increased block cracking was noted as part of the extension of existing cracks. There was increased diagonal cracking throughout the test specimen, with the yield line cracks opening to 1.0mm. The cracks at the bases of return walls A and B were 0.8mm and 0.3mm respectively. These cracks were visible but unmeasurable when the specimen was unloaded. The maximum crack width above the face window was 0.45mm. The base crack was measured to be 4.7mm wide, closing to 2.5mm.

Positive 4% drift

The initial pressure was 6.5kPa. A residual deformation of 36.3mm was observed, and when the reload pressure was 6.0kPa. The residual deformation was 25.3mm. The high residual deflection of the first loading may indicate some pressure remained in the system after unloading. As the tangent stiffness at low loads is very low, a small pressure can make a large change in deflection.

Starter bar strains increased greatly during this loading cycle, as can be seen in Figure 6.19. A maximum value of 12486 microstrain was recorded, 4.7 times the previous maximum. This great increase could be attributed to yield penetration caused by the large base rotations. If the original yield location was just above or below the strain gauge, it wouldn’t register the yield until strain hardening set in at the original yield site and the yield penetrated under the strain gauge. Maximum return wall starter bar strains recorded were 931 microstrain (A) and 709 microstrain (B).

Maximum return wall deflections were 4.3mm (A) and 3.0mm (B). At 4% drift the main wall still showed a smooth variation of displacement along it’s length, as seen in Figure 6.15. The first yielding of the bond beam was recorded, with 1514 microstrain recorded above the corner window.

Again, the increase in cracking consisted of the extension and opening of existing cracks. Cracks at the intersection of the main wall and return wall B reached 4.0mm wide, and these cracks closed very little when load was removed. The yield line crack below the
face window widened to 5mm wide and the out-of-plane discontinuity across the crack was also 5mm. The crack at the base averaged 8.0mm wide. The wall is 2400mm high and 190mm thick, so if it rotated as a rigid body through 96mm top deflection the expected crack width would be $96 \times 190 / 2400 = 7.6 \text{mm}$. The similarity between these two values confirms that the wall rotates as a rigid body about the base with very little curvature along its height.

Negative 4% drift

-96mm displacement at the centre of the bond beam required a pressure of 6.2kPa. When the load was removed a residual deformation of 26.8mm remained, with the required reload pressure being 5.6kPa. When this was released a 29mm residual deformation remained.

The vertical steel at the yield line crack below the face window was registering 3896 microstrain, 2.6 times the yield strain. The maximum recorded strain in starter bars was 11814, 2138 and 1723 microstrain in the main wall and return walls A and B respectively. The maximum strain in the bond beam was 2067 microstrain at the intersection of the main wall and return wall B.

Again a smooth variation of bond beam displacement was seen, as illustrated in Figure 6.16. Maximum return wall deformations were almost at the limit of instrumentation travel – 23.6mm (A) and 20.9mm (B) outwards.

The average base crack width was 7.5mm, again confirming the rigid body assumption. The horizontal crack below the bond beam in return wall A opened to 0.35mm wide, with the free end of the return wall deflecting 23.6mm. This suggests the increased crack width is due to the bond beam providing additional restraint to the return wall through torsion.
Positive to failure

As the travel of the central potentiometer was less than the expected wall displacement, a 500mm linear potentiometer was strung to the centre of the bond beam to give an expanded instrumentation range. The wall was then loaded constantly to failure.

Due to the dimensions of the airbags, at deflections past 200mm the pressure given by the instrumentation did not accurately reflect the pressure applied to the wall, as the gap between the loading frame and the wall was too large for the airbags to span. Because of this, the load-deflection plot shows deflection out to 200mm, even though the first major failure of the wall did not occur until 231mm. A shearing failure occurred along the yield line between the face window and return wall B, and significant damage resulted, with large displacements suddenly occurring. The wall was loaded beyond this point, but because the pressure loads were indeterminable the results are not admissible as part of a hysteresis response. At the end of loading the top of the wall had deflected over 1m and showed no signs of collapse. However in a realistic situation there would be an external vertical load on the wall that could cause collapse at such high deflections.

6.4.1 Strain Distributions

The data logging system used recorded the strain distributions in the starter bars, vertical steel and bond beam at every loading level. It was observed that up to 2% drift the central starter bars registered higher strains than those at the ends of the wall. This is because they are further from the restraining influence of the return walls and the wall is more free to rotate vertically in the middle. At 3% and 4% drift in both directions the strain in the starter bars was very high on either side of the face window and directly below where the yield lines intersected the bond beam. It is unknown why these high strains were recorded during these cycles.

Strain distributions in vertical bars were very similar to those observed in Zhang’s research. Below 2% drift in both directions the strain in the vertical bars above the base was always less than 40 microstrain. At 4% drift the maximum strain recorded in vertical steel away from yield lines was 497 microstrain, one third of the yield strain. This was at the centre of the wall, at the level of the bond beam. It is thought that the opening of the
horizontal portion of the diagonal cracks in the bond beam caused vertical elongation of the steel at this point.

Where the yield lines crossed the vertical steel, the wide cracks at high wall displacements caused yielding. The vertical bar on the return wall B side of the face window yielded at the height of the yield line crack during the −2% cycle. At −4% interstorey drift, the strain gauge was recording 3896 microstrain. The vertical bar beside the corner window yielded during the +2% cycle, and registered 1976 microstrain at the end of testing. These two locations were the only sites where the vertical steel was recorded as yielding.

Before yielding, the strain gauges in the bond beam showed negligible strains attributable to instrumentation errors. Beyond yielding the bond beam strains increased, but they were always less than the starter bar strains for each load increment. The maximum recorded bond beam strain excluding the corner details was 1514 microstrain during the +4% loading cycle recorded above the corner window. The first yield in a corner-detail bar occurred in the −3% cycle, and the strain during the −4% cycle was 2067 microstrain. Generally the strains in the bond beam for positive and negative loading to the same interstorey drift were within 15%. This indicated that the clip bar detail functioned effectively. In contrast, without the clip bar Zhang recorded a typical difference of 40% between positive and negative loading bond beam strains, due to different opening and closing moment capacities around the corners.

Strain distributions confirm that before first cracking most of the load is taken by vertical cantilever action with little contribution from the return walls. When the wall cracked, it became more flexible at the base and the bond beam began to take up load. When the first starter bar yield occurred, the wall became even more flexible at the base, and the bond beam strains increased significantly.

### 6.4.2 Curvature Distributions

In the +4.8kPa cycle the DEMEC points were read for the first time to obtain the strains and curvatures of the bond beam, and at various locations in the return wall. These points
were read for every post-elastic cycle from +4.8kPa onwards. Bond beam curvatures have been plotted as Figure 6.26 and Figure 6.27.

The bond beam curvature distribution highlights the concentration of curvature near the ends of the wall where the yield lines intersect the bond beam, and above the windows. The reversal of curvature in the return walls shows these walls are resisting the deformation of the main wall. The reversal of curvature at the ends of the main wall confirms the moment resistance about the main wall – return wall interfaces. The relatively small curvature at the centre of the wall suggests that this portion is behaving as yield line theory predicts and is deforming as a rigid body.

The curvatures of the free end of the return walls about a horizontal axis have been plotted as Figure 6.28 to Figure 6.31. In these it can be seen that the return walls have gone into double curvature. This means that an external moment must be applied to the top of the return wall by torsion from the bond beam. From the relative magnitude of the positive and negative curvatures, this moment appears to be of similar size to that applied at the base.

DEMEC points were installed to allow the examination of the level of in-plane bending of the return walls. The results obtained were within the margin of error of the instrumentation. While no relationship between load and in-plane curvature could be found, it can be stated that the in-plane curvatures in the return walls are very small.

The rosettes on the return walls did not provide the information on shear and hence torsion that was hoped for. Due to the very low strains involved, instrumentation errors obscured any trend and prevented valid conclusions being drawn.
Figure 6.26 – Bond beam curvatures due to positive loading  
(Plotted on compression side)

Figure 6.27 – Bond beam curvatures due to negative loading  
(Plotted on compression side)
Figure 6.28 – Return wall A free edge curvatures due to positive loading

Figure 6.29 – Return wall A free edge curvatures due to negative loading
Figure 6.30 – Return wall B free edge curvatures due to positive loading

Figure 6.31 – Return wall B free edge curvatures due to negative loading.
7 Discussion

7.1 Validity of Testing

The value of any research is doubtful unless it can be shown to apply to the situation in which it will be applied. The current research must be shown to be applicable to a wall under seismic loading.

7.1.1 Validity of Loading Pattern

The use of air bags to provide the out-of-plane loading on the face of the wall meant that the pressure applied to the wall was constant everywhere. This is not consistent with the current New Zealand loadings code, NZS 4203: 1992, which for the equivalent static method suggests a linear increase of loading with height, to simulate first mode effects, and a point load at the top of the wall to simulate higher mode effects.

Single-storey reinforced masonry has been found to respond to seismic activity predominantly in the first mode. Because of this, neglecting the point load at the top of the wall, which should correspond to 8% of the lateral equivalent load, should have little influence on the validity of the testing.

By equating moments at the base of the wall under the triangular and uniform loading patterns, the uniform loading pattern can be shown to be non-conservative relative to seismic loading. The reaction forces to these different loadings patterns with the same total load on an unpropped and propped cantilever can be seen in Figure 7.1.

![Reactions due to uniform and triangular pressure distributions](image)

Figure 7.1 – Reactions due to uniform and triangular pressure distributions
Where

\[ P_{\text{uni}} = \frac{1}{2}wh - \frac{M_{\text{yield}}}{h} \quad \text{and} \quad P_{\text{tri}} = \frac{2}{3}wh - \frac{M_{\text{yield}}}{h} \]

At low loads, the wall responds more as an unpropped cantilever, and the load is resisted by the base moment. As this moment is 1/3 higher for the triangular loading distribution, and the bond-beam deflection is a function of this moment and the loading distribution, this deflection could be up to 1/3 higher under seismic loading when compared to the testing behaviour.

At higher deflections when the load is resisted as a propped cantilever, the load is primarily resisted by the bond beam, with the moment at the base remaining fixed at the yield value, determined from testing to be 10.4 kN.m/m. At the ultimate load of 5.8kPa the point load reaction is 4.95kN for the triangular loading distribution and 2.63kN for the uniform loading distribution. This indicates that the load on the bond beam under seismic loading would be 1.9 times higher than in an experiment. However, it must be noted that strain gauges on the bond beam during experimentation indicated it was well within its strength capacity, so this extra load may not have significant detrimental effects on wall performance. At post-yield deflections the vertical steel along the yield lines will be resisting the applied load. This and other load-resisting mechanisms such as torsion in grouted cells have not been taken into effect in this simple comparison, and their exclusion has made the difference between the bond-beam reactions under the two loading patterns more pronounced than would really be the case.

No external axial load was applied to the test specimen. This was considered appropriate, because in single storey construction the axial force on the wall would be small, and this combined with the wall’s large axial load capacity results in a very low axial load ratio in practice. In under-reinforced walls, axial load tends to increase moment capacity, especially for centrally reinforced walls such as the test specimen.

7.1.2 Validity of Static Testing

Conducting static tests offers a number of advantages over dynamic testing. The ability to start and stop static testing allows measurements and observations to be made that
would not be possible in a dynamic test. The progress of cracking across a wall can be recorded, and curvature readings can easily be made using a DEMEC gauge. The ability to control loading and deflection directly allows easier evaluation of yield points and all facets of hysteretic response.

However, the dynamic behaviour of the wall subject to seismic excitation must be determined from these static tests. Work by Mahin [1972] found that there is no basic difference in behaviour under static and dynamic loading. More detailed work by Shah and Chung [1986] on the effect of loading rate on wall behaviour found that a wall loaded over 2 seconds would develop an ultimate strength 20-25% greater than if the wall had been loaded over 6.5 minutes. The wall under fast loading also showed less distributed cracking but instead a wider crack at the wall’s base. They concluded that results obtained from static testing could give conservative values for ultimate strength and hysteretic behaviour relative to that that would be exhibited under seismic loading.

7.2 Wall Behaviour

7.2.1 General Behaviour

Under Serviceability Limit State loading of 0.26kPa no cracking was observed. The maximum recorded deflection was 0.83mm at midspan along the bond beam. No strain was recorded in the starter bars or the bond beam.

Under Ultimate Limit State loading of 1.6kPa no cracking was observed, and a maximum deflection of 2.2mm was recorded when the wall was loaded in the negative direction. The maximum starter bar strain also occurred in the negative direction, with 203 microstrain being recorded in the central starter bar. The wall remained uncracked under positive ULS loading, but a very fine 3.6m long crack formed in the mortar joint at the base of the wall under negative ULS loading.

At the start of the test there were a few observable shrinkage cracks above both the corner and face window, which were all <0.1mm wide. The first cracks due to loading occurred at +2.2kPa and −1.6kPa. At low levels of load the only crack of substantial length was at the base of the wall. All cracks closed fully when load was released until a load level of
4.8kPa was applied, which left some very fine cracking visible after the load was released.

Return wall cracking was first observed at +4.8kPa and −4.6kPa. These cracks were found to be consistent with the in-plane, out-of-plane and torsion loads applied to the return walls by the main wall bond beam.

Maximum crack widths at 1% drift were 1.75mm at the base of the wall and 0.4mm in the bond beam. Hairline cracking through some blocks was observed at this load level. At 2% drift the maximum base crack width was 3.5mm, and the widest crack in the wall was 1.5mm wide. Cracks in blocks began to open up, with the widest measured at 1.2mm.

The width of cracks observed was considerably less than that observed in previous research [Singh, Zhang]. This was attributed to the more widespread cracking, with wall rotation not being concentrated across one crack. The progression of cracking can be seen in Appendix 3.

The blocks above the corner window performed excellently, showing less cracking and smaller crack widths than observed on Zhang’s specimen that did not use the clip-bar detail. This confirms that this reinforcing detail provides enhanced moment capacity around bond-beam corners.

Yield was determined from the load-deflection plot to have occurred at a load of 3.6kPa and a deflection of 8.54mm. At the point of first yield it was observed that the wall’s stiffness began to degrade. This degradation continued through further loading cycles, with the wall having very little stiffness when unloaded, resulting in the pinched nature of the post-elastic hysteresis curve. For the load cycles of 1% inter-storey drift and beyond it was found that the wall displayed similar stiffness’s on the unloading-reloading-unloading part of each cycle.

The load resisting capacity of the wall continued to climb with each load cycle. The load at yield was 3.6kPa, but at 200mm deflection on the final testing cycle the wall had reached 8.2kPa. Past 200mm the recorded load continued to climb, but as the airbags were reaching the limit of their travel these figures do not accurately represent the load
applied to the full face of the wall. The first catastrophic damage occurred at the intersection of the main wall and return wall B at a midspan deflection of 231mm, which represents an inter-storey drift ratio of 9.6%.

7.2.2 Load-Deflection Response

The load-deflection hysteresis loops for the test specimen are illustrated in Figure 6.3 and Figure 6.5. It can be seen that before yield there is no degradation in stiffness. Beyond yield the hysteresis loops are very pinched, with a region of zero tangent stiffness at low loads. However, while secant stiffness decreased with each load cycle the tangent stiffness at high loads varies little between cycles. Load carrying capacity increased with each load cycle.

Zhang [1998] suggested an explanation for the shape of the hysteresis curve. Initially the wall is uncracked and the full section acts to resist out-of-plane bending. This accounts for the unchanged stiffness in the pre-yield load cycles. Once the wall has cracked and the starter bars yielded, the section resisting the out-of-plane action is reduced from the gross section to the transformed section, accounting for the greatly reduced stiffness of the load-deflection envelope past yield. This transformed section consists of the reinforcing in tension and the masonry shell in compression.

Once the crack has opened sufficiently, no tension may be transferred across the concrete. To maintain equilibrium, the steel must be either in tension or unstressed. Yielding of the steel under load will cause it to elongate, and when the loading is reversed the wall will rock with very low stiffness on the elongated reinforcing until the crack closes and the masonry shell begins to take compression. This mechanism would occur in the same manner for all post-elastic loading cycles, which agrees with the observation that the tangent stiffness is similar for all post-elastic cycles. Figure 6.8, a plot of the +2% loading cycle, illustrates this.

The hysteretic damping ratio $\zeta$ is defined by $\zeta = \frac{A_h}{4\pi A_c}$ where $A_h$ is the area enclosed in the loop and $A_c$ is ½ the area of the smallest box that can fully enclose the loop. $A_h$ was similar for the cycles between 1% drift and 4% drift, and $A_c$ increased almost linearly
with deflection. This indicates that the damping ratio of the wall post-yield decreases almost linearly with deflection.

Partially grouted masonry walls have been shown to display ductile behaviour with pinched hysteresis loops both in-plane [Brammer, 1995] and out of plane by this and previous testing. The stiffness degradation of a wall loaded past yield will cause an increase in fundamental period (for harmonic motion, $T \propto 1 / \text{stiffness}^2$). This shift in period may increase the peak spectral acceleration of the wall [Paulay, 1992].

### 7.2.3 Opening Effect

The finite element analysis revealed that at low deflections the major load carrying mechanism is vertical cantilever action, with the bond-beam playing contributing little strength. This suggests than any openings will not affect the cracking load of the member, as under vertical cantilever action it does not matter if the vertical cantilever in question is part of a complete wall or between two openings.

This is confirmed by the results of the first three test specimens tested at University of Canterbury, which display very similar behaviour out to 10mm deflection, and don’t begin to perform differently until the walls have cracked and the starter bars begun to yield. The load-deflection envelopes for this and the previous tests have been plotted as Figure 7.2 and it can be seen that the envelopes from previous tests do not begin to significantly diverge until 7mm midspan deflection, at the point of first rebar yield.

![Figure 7.2 – Load deflection envelopes for recent research at Canterbury](image_url)
It is observable that the test specimen constructed for this research project was significantly more flexible at low levels of load, but had similar behaviour at higher loads. Because at lower loads it is the starter bars that govern behaviour, it would seem likely that some issue with the starter bars caused this loss of stiffness. However, when at the end of testing the starter bars were inspected, no pull-out was determined to have occurred, and the vertical cells were filled, giving a good bond with the vertical steel. The reason for the lack of stiffness is unknown.

The variability in strength of mortar bedding joints (Zhang [1998] found mortar bond strength at testing to vary from 22.1 kPa to 45.4 kPa over six tests) is far more likely to cause a difference in cracking loads between two specimens than the location, size and number of openings.

Openings located at the wall ends and along the yield line reduce the capacity of the wall the most. The window opening in the centre of the face of Zhang’s second specimen did not affect the load resisting capacity but did reduce the load applied, which enhanced the apparent strength of the specimen. Table 7.1 lists the reductions in load and area of the recent test specimens at the University of Canterbury. It is considered that between the three tests with openings the most non-conservative geometric layouts for windows have been tested, and found to perform well. Any other arrangement of windows and doors will result in a higher ultimate strength, and better overall performance.

<table>
<thead>
<tr>
<th>Type of Wall</th>
<th>Ultimate Load</th>
<th>Load Reduction</th>
<th>Area Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Openings [Singh, 1998]</td>
<td>7.6kPa</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td>Two Door Openings [Zhangh, 1998]</td>
<td>5.4kPa</td>
<td>29%</td>
<td>15%</td>
</tr>
<tr>
<td>Door and Window Openings [Zhang, 1998]</td>
<td>6.6kPa</td>
<td>13%</td>
<td>22%</td>
</tr>
<tr>
<td>Window Openings [This Test]</td>
<td>5.8kPa</td>
<td>24%</td>
<td>9%</td>
</tr>
</tbody>
</table>
7.2.4 Reinforcing Details

The clip bar installed in the bond beam increased the moment resisting capacity of these corners and reduced post-elastic damage. This was particularly evident above the corner window, which has shown less damage than the bond beam above the corner window in Zhang's second test specimen, which did not contain this detail. Figure 7.3 shows the bond beam above the corner window at +4% drift.

![Figure 7.3 - Bond beam above window at +4% drift](image)

The lapping of bars at the corner windows as used in this test specimen would not be suitable for general use. During grouting the specimen was rodded far in excess of code requirements in an attempt to ensure no voids around the steel, yet when the wall was demolished it was found that there were voids under the corner steel lap zones. This has been attributed to steel congestion at the lap site, with the three bars blocking off a large portion of the available cavity width. These voids at the lap locations could severely compromise the transfer of load between bars. To remediate this an alternative detail has been proposed in Figure 7.4, which should prevent voids forming at lap locations if proper care is taken during grouting.
7.2.5 The Bond Beam as a Catenary

It was found in the last loading half-cycle, where the wall was pushed to a deflection of beyond 200mm, that the wall’s strength was not degrading, but in fact as the deflection increased the wall’s recorded load resisting capacity also increased. This has been attributed in part to the steel in the bond beam assuming a curved shape and carrying the applied load through tension rather than bending. Figure 7.5 illustrates this.

This was also noted in Singh’s research [1998], where the wall without openings displayed a steady increase in strength until testing ended at a midspan deflection of 287.5mm, which corresponds to a drift level of 8.3%.

However, as the drifts associated with wall strengthening are very high there is little practical application for this considerable increase in strength, as any connected component, be it structural or non-structural, would be severely damaged. However the
wall itself will not fully collapse, which would be important from a preservation of life perspective during a very severe earthquake.

7.2.6 Above Window Weakness

During demolition it was noted that due to the shape of the blocks above the windows, there was a section of weakness every 200mm. This is because the half-end closer (20.12) blocks used have a very thick base, and the mortar does not completely cover this base. As the space between blocks is only around 10mm wide, the grout does not penetrate this gap, as shown in Figure 7.6.

![Diagram of a block with mortar and an unfilled area](image)

**Figure 7.6** – Half-end closer / lintel block showing area unfilled by mortar or grout

This leaves an area of air inside the bond beam, which reduces its strength considerably. Preliminary calculations have indicated that this void could reduce the uncracked stiffness and strength of the bond beam by 4%. As the void and the compressive stress block do not overlap, the void should not affect post-cracked behaviour directly, however it may interfere with transfer of load between the reinforcing steel and compression block, reducing the bond beam’s moment capacity slightly. This reduction in bond beam moment capacity may affect the overall load resisting capacity of the wall.

Possible solutions would include making sure that the ends of these blocks are covered with mortar, and using “u” blocks with bases the same thickness as a normal block. For
similar reasons, open-end blocks should be used in the body of the wall, as closed-end blocks will have a mortarless gap between their end faces.

7.2.7 Grout Penetration

When the braces were removed from the windows after grouting, a small hole was made in the mortar beneath both windows. This was probed and it was found that the grout had not fully penetrated beneath the lower layer of bond-beam steel above the corner window. This was attributed to the steel congestion, with three bars running in each course at this point. The corner cell itself, which had been found difficult to fill with a different bar layout in previous research, was completely filled. Before testing, holes were drilled to ascertain the extent of this void, and it was filled under pressure with an expansive grout mixture. This grout mixture was tested and found to have similar properties to the grout initially used to fill the wall, hence no changes to analysis were necessary.

7.2.8 Return Wall Crack Patterns

The return walls had no load applied directly. All loads in the return walls were due to the transfer of forces and moments at the main wall / return wall intersection. Due to the level of cracking at this interface in the unreinforced sections below the level of the bond beam, and their low load transfer capacity when cracked, these actions have been assumed to act predominantly at the level of the bond beam.

These actions have been separated into three independent components.

1. A moment applied to the top of the return wall due to the rotation of the main wall bond beam
2. An in-plane load applied to the top of the return wall in the direction of airbag loading due to the reaction of the bond-beam beam action.
3. An out-of-plane loading applied to the top of the return wall due to bond beam growth (outwards force) and catenary action (inwards force).

The principal stress directions and theoretical crack patterns for these actions has been plotted as Figure 7.7. The direction of the applied load will determine the direction of the applied action; for the example negative loading has been used. It can be seen that for some areas of the wall one action will cause compression while another may cause
compression. This interference of stress fields results in less distinct cracking patterns on the test specimen, as can be seen in Appendix 3.

![Diagram showing principal stress orientations and cracking patterns](image)

Figure 7.7 – Theoretical return wall principal stress orientations and crack patterns

By comparing these patterns with those in Appendix 3 some of the interactions between the different stress fields can be seen. For example, the base crack due to in-plane bending will not form on the side which is in compression due to out-of-plane bending. Some crack patterns are not substantially evident for one loading direction, indicating that loading in that direction does not induce the action on the return wall at a high enough level to cause cracking. For example, the cracks in the upper courses due to out-of-plane loading and bond beam restraint are only visible in return wall A at the cycles −2% and beyond.
7.3 Seismic Performance of the Test Specimen

7.3.1 Load Resistance Capacity

As the load carrying capacity did not decrease as load increased, the traditional definition of ultimate strength is inapplicable. The ultimate strength will instead be defined as the load at which the wall reached the codified deflection limit from NZS 4230: 1992. This limit of 2% is equivalent to 48mm for a 2.4m high wall. At 48mm deflection the load on the wall was 5.8kPa. The yield and ultimate load capacities from the previous research at Canterbury has been tabulated in Table 7.2 below.

<table>
<thead>
<tr>
<th>Type of Wall</th>
<th>First Yield Load (Tangent)</th>
<th>First Yield Load (Bar Yield)</th>
<th>Ultimate Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Openings [Singh, 1998]</td>
<td>3.6kPa</td>
<td></td>
<td>7.6kPa</td>
</tr>
<tr>
<td>Two Door Openings [Zhang, 1998]</td>
<td>3.3kPa</td>
<td>4.5kPa</td>
<td>5.4kPa</td>
</tr>
<tr>
<td>Door and Window Openings [Zhang, 1998]</td>
<td>3.4kPa</td>
<td>5.5kPa</td>
<td>6.6kPa</td>
</tr>
<tr>
<td>Window Openings [This Test]</td>
<td>3.6kPa</td>
<td></td>
<td>5.8kPa</td>
</tr>
</tbody>
</table>

The yield load was calculated for the three previous test walls by the hysteretic tangent method used in this research to give a valid comparison. As this data was scaled off a graph, there may be a slight error inherent in the values for the previous tests. Zhang identified the load to cause all starter bars to yield, but instrumentation difficulties prevented this on the current project.

The difference between the highest and lowest yield load defined using the tangent method is 8.3%, compared with the difference between the highest and lowest ultimate
load, 28%. From this it is evident that opening size and location affect the ultimate performance of the wall much more than the serviceability performance.

Zhang [1998] identified the maximum seismic load $F_{ph}$ on these walls to be 3.5kPa, using NZS 4230: 1992 and assuming the walls behaved as "parts and portions" and the most severe conditions for site location and conditions. This load is due to the self-weight of the walls only. In Table 7.3 this maximum load, and the two next highest loads for different conditions, has been compared with the capacities of the experimental walls, and the factor of safety determined.

<table>
<thead>
<tr>
<th>Type of Wall</th>
<th>$F_{ph}$=3.5kPa</th>
<th>$F_{ph}$=2.9kPa</th>
<th>$F_{ph}$=2.5kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Openings [Singh, 1998]</td>
<td>2.2</td>
<td>2.6</td>
<td>3.0</td>
</tr>
<tr>
<td>Two Door Openings [Zhang, 1998]</td>
<td>1.5</td>
<td>1.9</td>
<td>2.2</td>
</tr>
<tr>
<td>Door and Window Openings [Zhang, 1998]</td>
<td>1.9</td>
<td>2.3</td>
<td>2.7</td>
</tr>
<tr>
<td>Window Openings [This Test]</td>
<td>1.7</td>
<td>2.0</td>
<td>2.3</td>
</tr>
</tbody>
</table>

In residential construction a partially grouted masonry wall would typically be topped with a lightweight timber roof. Zhang [1998] determined that the maximum weight of this roof on the wall would be 300kg/m, and that assuming a flexible diaphragm, that half of the seismic load due to this weight would be transferred to the bond beam and half would be transferred directly to the return walls.

NZS 4230: 1992 states that this roofing could be considered a "part and portion" for the purposes of evaluation of seismic load. Zhang [1998] calculated the maximum seismic face load coefficient $C_{pi}$ as 1.1, and this has been used along with the next two highest values for $C_{pi}$ to determine the factor of safety for the test walls under seismic load with a light timber roofing system. This has been tabulated in Table 7.4.
Table 7.4 – Safety Factors for the Test Walls with Light Timber Roofing

<table>
<thead>
<tr>
<th>Type of Wall</th>
<th>$C_{p1}=1.1$</th>
<th>$C_{p1}=0.9$</th>
<th>$C_{p1}=0.78$</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Openings [Singh, 1998]</td>
<td>1.4</td>
<td>1.6</td>
<td>1.9</td>
</tr>
<tr>
<td>Two Door Openings [Zhang, 1998]</td>
<td>1.0</td>
<td>1.2</td>
<td>1.4</td>
</tr>
<tr>
<td>Door and Window Openings [Zhang, 1998]</td>
<td>1.2</td>
<td>1.4</td>
<td>1.7</td>
</tr>
<tr>
<td>Window Openings [This Test]</td>
<td>1.1</td>
<td>1.3</td>
<td>1.5</td>
</tr>
</tbody>
</table>

7.3.2 Available Ductility of the Test Wall

The test results showed a ductile behaviour which "pinched" during high displacement cycles. The walls were capable of sustaining high displacement before collapse, with increasing load capacity as deflection increased. Structural yield was determined by the tangent method to have occurred at 8.5mm, but no clearly defined yield point was observed. The deflection at the ultimate load was the codified 2% interstorey drift, 48mm. From this it has been determined that the available displacement ductility of the test specimen was 5.6.

7.3.3 Load Resistance Capacity based on Available Ductility

For this wall, the ultimate applied seismic load with no roof calculated elastically is less than the yield load of the structure. In the design earthquake the wall will not yield and the load resistance capacity based on available ductility will not be a relevant measure of wall capacity. The wall can be expected to behave elastically and designed for out-of-plane effects.

The addition of a light-timber roof increases the equivalent applied seismic loads to above the yield load but below the ultimate load. As the inclusion of ductility in NZS 4203 : 1992 reduces the applied load, the ultimate capacity of the wall is greater than the applied load either including or excluding ductility effects. Thus the wall with a light timber roof can also be designed to carry loads based on an elastic response, as this will be a conservative design.
If a designer wished to take the load-reducing effects of ductility into account, several difficulties would have to be surmounted. The ultimate out-of-plane behaviour of the wall is governed by deflection limits, not by strength limits. Excessive deflection and not collapse hence define failure. The factor of safety against excessive deflection for the wall utilising it’s available ductility cannot be calculated using a load-based code like NZS 4203: 1992. This code requires knowledge of the period of the structure to determine the base horizontal coefficient. The effect of the period shift due to loss of stiffness is unknown, but will almost certainly be so low that the structure does not move out of the <0.3s constant coefficient region.

To determine the behaviour of a similar wall in an earthquake that causes an equivalent seismic load bigger than the structural yield load, a different analysis technique will have to be used. A time history analysis incorporating the hysteretic shape recorded in testing would allow the determination of whether or not the wall fails by exceeding 2% interstorey drift. This would also permit the investigation of the period shift due to change in stiffness.
8 Conclusions

The aim of this project was to investigate the out-of-plane behaviour of a partially grouted masonry wall under simulated seismic loading. The specimen tested was 9.4m long by 2.4m high, with 1.8m long return walls at each end giving it a “C” shaped plan. Windows were located to reduce the capacity of the wall – one in the corner which previous research had shown to reduce the capacity of the wall, and one along the 45° yield line. The wall contained the minimum reinforcing specified by NZS 4229: 1999, and exceeded the code’s maximum length between return walls. 2D16 bars were laid in the top two courses as a bond beam, and the vertical reinforcing consisted of D12 bars at 800mm centres. Clip bars were installed at the bond beam corners in a successful attempt to reduce damage at these locations. Theoretical studies to predict the behaviour of the wall were made. The conclusions that have been drawn from the research are summarised in this chapter.

8.1 General Behaviour

There were very few shrinkage cracks visible before the start of testing. Those cracks that were visible were all fine and short. The first cracks were observed in the mortar joints at the base of the wall at +2.2kPa and −1.6kPa.

Yield was determined to have occurred at a load of 3.6kPa and a deflection of 8.54mm. This was determined from the intersection of the pre- and post-yield hysteretic curves.

The ultimate load capacity of the test specimen was 5.8kPa. This was defined using the codified deflection limit of 2% interstorey drift, as specified by NZS 4229: 1992. This deflection limit is equivalent to a displacement ductility of 5.6 in this test.

The test specimen showed ductile behaviour with a “pinched” hysteretic response. Load carrying capacities increased as deflection increased, with large deflections (230mm at midspan along the bond beam) occurring before catastrophic damage occurred in the bond beam near return wall B. Once this damage had occurred the wall did not collapse, but it’s structural integrity was severely compromised.
8.2 Performance in SLS and ULS

Under serviceability limit state (SLS) loading of 0.26kPa the deflection of the bond beam at midspan was 0.8mm for loading in both directions. No cracks were observed and no strain recorded at this loading.

Under ultimate limit state (ULS) loading of 1.6kPa the deflection of the bond beam at midspan was +1.7mm and −2.2mm for positive and negative loading. The maximum recorded strain in a starter bar was 203 microstrain, 14% of the yield strain. No cracking was observed under positive loading, but under negative loading a very fine 3.6m long crack formed in the mortar at the base of the wall.

Ultimate load capacity was taken at 2% drift, the maximum specified in NZS 4229: 1992. At this drift the load on the wall was 5.8 kPa. The base crack was measured at 3.1mm wide, and a 1.2mm wide crack had formed in the bond beam above the corner window.

8.3 Available Displacement Ductility

Available displacement ductility was calculated using the yield displacement and 2% interstorey drift limit as 5.6. The wall showed no reduction in load carrying capacity even at high interstorey drifts.

8.4 Ultimate Out-of-Plane Strength

The ultimate load resisting capacity of the test specimen was 5.8kPa at 2% interstorey drift. This is 1.7 times higher than the maximum codified seismic face load of 3.5kPa, which assumes flexible soils in the Wellington area. This load is due to wall self-weight only, and the addition of a light-timber roof reduces this factor of safety to 1.1.

The wall was found to behave elastically at the ultimate seismic load. With the addition of a light-timber roof the wall will behave inelastically, but should be able to resist the applied load, with no load reduction due to ductility being necessary.
The ultimate capacity of the test specimen was determined under uniform pressure loading, unlike the distribution of seismic loading. The effect of different wall configurations and the contribution to strength of the roof was not investigated. More research on the interaction of these components is necessary to determine the level of safety with these components attached.

8.5 Opening Effect

The location and size of openings in partially grouted masonry walls has been shown to have a significant effect on their ultimate out-of-plane strength. The tests showed that the reduction in ultimate load was 24% when compared with that of the wall without opening [Singh, 1998].

8.6 Reinforcing Details

The horizontal reinforcing in cells directly under the windows was hooked downwards 150mm instead of extending it 600mm into the adjacent cells. This simplified grouting considerably, and strain gauges installed on this steel showed it to have performed well.

The clip-bar detail used in the bond-beam corners performed excellently. Damage was reduced when compared with the previous test specimen with a corner window [Zhang, 1998] that did not use this detail. A modification to that used in the experiment is recommended to ensure grout penetration at the lap sites.

8.7 Analysis Techniques

A simple Finite Element model was constructed using ABAQUS. The wall was modelled as a solid shell, which is too stiff before yield but gave an excellent match with experimental behaviour after yield.

Traditional yield line analysis provided a conservative value for the ultimate strength of the wall. A Force-Deflection Yield Line Analysis was devised to take into account bond
slip and strain hardening, and this extended the range of accurate modelling, with a good match with experimental data out to 75mm drift.
9 Future Research

The walls performed excellently during testing, with ultimate loads higher than those expected in the current non-specific design code NZS 4229: 1999. Theoretical studies conducted also predicted these higher loads. This suggests that while the revision of NZS 4229: 1986 removed some of the conservatism of the code, the code requirements may still be too strict as applied to allowable wall lengths and bond-beam steel requirements.

Many factors untested in this research program can influence seismic behaviour, especially wall plan configuration and the mass, location and structural contribution of attached components. The quality of workmanship employed in the construction of a partially grouted wall can significantly affect its response. Further study in the following areas would improve our understanding of the performance of these walls in realistic situations:

1. The interaction of in-plane and out-of-plane response of a single storey masonry structure with a flexible roofing system would need to be specifically studied. The in-plane response of the structure and the flexible roofing system could cause a significant amplification to the out-of-plane response due to resonance effects.

2. Theoretical and experimental studies have shown that the corner window reduces the out-of-plane load resistance significantly. Static testing cannot determine its influence on the torsional rigidity of the structure. A dynamic test would investigate the performance of the corner window during an earthquake. A lightweight timber roof could be installed and the interaction studied.

3. In masonry residential buildings, timber frames are often used as internal partitions. The benefit obtained from timber partitions to the out-of-plane behaviour of these walls would be worth investigating.

4. It is assumed that the lateral seismic forces from the flexible light-timber roof transfers to the transverse walls by the tributary area, although this assumption is unsupported. The additional seismic out-of-plane load due to a flexible roof or additional storey on the masonry structure could be studied.
5. The seismic force level for a single storey masonry structure would need to be investigated carefully, as these structures generally have a short fundamental period, and fall into the constant acceleration and equal energy regions in design response spectra. With the data for hysteretic behaviour obtained it should be possible to conduct a computerised time-history seismic analysis. This would reveal any significant shifts in wall period as yielding occurred, which would be important as this period affects the base seismic coefficient for calculating equivalent seismic loads. It is possible that the inelastic response spectra are non-conservative for short-period structures with poor hysteretic loop shapes.

6. The effect of poor grouting, could be investigated by purposefully debonding bars, particularly with regard to the reduction in load resisting capacity. This reduction could be significant, and dangerously reduce the structure’s factor of safety.

7. A refined moment-curvature analysis of masonry strips in the horizontal and vertical directions would allow improved accuracy for finite element and yield line analyses. Similarly, improved modelling or testing of moment-rotation relationships for the base joint and bond-beam would allow better force-deflection yield line analysis.
10 References


12. Mahin, S. A., Bertero, V. V., (1972) “Rate of Loading Effects on Uncracked and Repaired Reinforced Concrete Members” EERC 72-9, University of California, Berkeley, 121p.
11 Appendix 1 – Test Specimen Plans and Details
All steel D12 except bond beam steel D16. All 90° bends 150mm.

R6 stirrups @ 200mm in bond beam above windows. R6 stirrups @ 600mm elsewhere.
Bondbeam Cross-section

Stirrup spacing -
200mm at lintels
600mm away from lintels

Bondbeam Corner Detail
All potentiometers mounted at the upper edge of the bond beam.
Upper bond beam course corner steel

Lower bond beam course corner steel

Strain Gauge Locations
13 Appendix 3 – Crack Propagation Record
Positive 2.2kPa loading

Positive 4.8kPa loading
Positive 17.5mm deflection

Positive 1% drift
Outside Elevation

Inside Elevation

Positive 2% drift

Outside Elevation

Inside Elevation

Positive 3% drift
Positive 4% drift
Outside Elevation

Inside Elevation

Negative 1.6kPa loading

Outside Elevation

Inside Elevation

Negative 1.9kPa loading
Negative 3.0kPa loading

Negative 17.5mm deflection
Outside Elevation

Inside Elevation

Negative 1% drift

Outside Elevation

Inside Elevation

Negative 2% drift
Outside Elevation

Negative 3% drift

Inside Elevation

Outside Elevation

Inside Elevation

Negative 4% drift
14 Appendix 4 – Finite Element Analysis Data
ABAQUS Finite Element Analysis input file

*HEADING
OUT OF PLANE COLLAPSE OF MASONRY TEST SPECIMEN 30/01/01 ROBERT BAXTER

*NODE, NSET=ALLNODES
101,0,0.
148,9400,0.
1301,0,2400.
1348,9400,2400.

*NGEN, NSET=ALLNODES
101,1301,100
148,1348,100
101,148,1
201,248,1
301,348,1
401,448,1
501,548,1
601,648,1
701,748,1
801,848,1
901,948,1
1001,1048,1
1101,1148,1
1201,1248,1
1301,1348,1

*NSET, NSET=FIXED
101,201,301,401,501,601,701,801,901,1001,1101,1201,1301
148,248,348,448,548,648,748,848,948,1048,1148,1248,1348
115,116,117,118,119,120,121,122,123,124,125,126,127,128
129,130,131,132,133,134,135,136,137,138,139,140,141,142
143,144,145,146,147,148

*ELEMENT, TYPE=S4R, ELSET=WALL
101,101,102,202,201
606,606,607,707,706
643,643,644,744,743
1101,1101,1102,1202,1201

*ELGEN, ELSET=WALL
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606,32,1,1,5,100,100
643,5,1,1,5,100,100
1101,47,1,1,2,100,100

*ELSET, ELSET=BBEAM, GENERATE
1101,1147
1201,1247

*ELSET, ELSET=SILL, GENERATE
501,506
537,543

*ELSET, ELSET=VSTEEL, GENERATE
101,501,100
105,505,100
106,1206,100
110,1210,100
114,1214,100
118,1218,100
122,1222,100
126,1226,100
130,1230,100
134,1234,100
137,1237,100
140,540,100
143,1243,100
147,1247,100
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  181,9
  * MATERIAL, NAME=A1
  * ELASTIC
  13676,0.15
  * CONCRETE
  9.,0.
  13.67,0.003
  * FAILURE RATIOS
  1.14 , .0599
  * TENSION STIFFENING
  1.,0.
  0.,2.E-3
  * REBAR, ELEMENT=SH ELL, MATERIAL=WALLST, GEOMETRY=ISOPARAMETRIC
    VSTEEL, 113.1,200,-4.5,2
  * REBAR, ELEMENT=SH ELL, MATERIAL=WALLST, GEOMETRY=ISOPARAMETRIC
    SILL, 113.1,200,-4.5,1
  * REBAR, ELEMENT=SH ELL, MATERIAL=WALLST, GEOMETRY=ISOPARAMETRIC
    BBEAM, 402.1,200,-4.5,1
  * MATERIAL, NAME=WALLST
  * ELASTIC
    2.E5
  * PLASTIC
    309
  * BOUNDARY
    FIXED, 1,6
  * RESTART, WRITE
  * STEP, EXTRAPOLATION=LINEAR, INC=100, NLGEOM
  * STATIC
    0.01,1.
  * DLOAD
    WALL,BZ, 0.001
  * END STEP
Cells with horizontal D16 rebar at 200mm centres

Cells with horizontal D12 rebar at 200mm centres

Cells with vertical D12 rebar at 200mm centres
Moment about vertical axis at failure
(Contours with crosses are zero-moment)

Moment about horizontal axis at failure
(Contours with crosses are zero-moment)

Deformed shape at failure
15 Appendix 5 – Test Specimen Construction Photographic Record
Polystyrene pods laid out in formwork. Note holes in pods located at reaction frame anchor points.

Mesh laid on polystyrene pods supported by 20mm mesh chairs.
Pumping concrete to fill the Ribraft floor slab.

Levelling the floor slab.
Strain gauge installation on bond beam steel.

Laying the first blocks over the starter bars.
Temporary supports at windows allow the bond beam to be laid.

Blocklaying complete. Inspection ports in cells to be grouted visible.
Grouting the bond beam and vertical cells.

Rodding the bond beam to remove air voids.