Numerical modelling and testing of concrete walls with minimum vertical reinforcement

Y. Lu, R.S. Henry & Q.T. Ma

Department of Civil and Environmental Engineering, The University of Auckland, Auckland.

2014 NZSEE Conference

ABSTRACT: During the 2010/2011 Canterbury earthquakes, several lightly reinforced concrete (RC) walls in multi-storey buildings formed a limited number of cracks at the wall base and was found with fractured vertical reinforcements. This unexpected behaviour raises a question regarding whether RC walls designed according to current minimum vertical reinforcement requirements can exhibit sufficient ductility during earthquakes. A detailed experimental investigation is currently underway to verify the seismic performance of RC walls with current code specified minimum vertical reinforcement. A test setup has been developed to subject the lower portion of a RC wall specimen to loading that is representative of a multi-storey building. Prior to the experimental tests, a series of numerical analyses were conducted to predict the response of the test walls, and for calibration, a lightly RC wall that was damaged during the Canterbury Earthquakes. The numerical analysis successfully replicated the observed failure mode of the lightly RC wall. Push-over analysis results also indicated that the test walls designed in accordance with NZS 3101:2006 minimum vertical reinforcement requirements may be susceptible to limited flexural cracking and premature reinforcement fracture. Furthermore, the drift capacity of RC walls with minimum vertical reinforcement improved as the aspect ratio or axial load ratio was increased.

1 INTRODUCTION

Minimum reinforcement requirements for reinforced concrete (RC) walls are imposed by most concrete design standards worldwide. Requirements for longitudinal, or vertical, reinforcement are not only imposed to mitigate shrinkage and temperature effects, but also intended to prevent non-ductile failure modes. If insufficient vertical reinforcement is provided in RC walls, the tension force generated by the reinforcing steel may not be large enough to develop secondary flexural cracks in the surrounding concrete. This behaviour can lead to a limited number of cracks in the plastic hinge region at the wall base, reducing the inelastic deformation capacity, and resulting in premature fracture of vertical reinforcement.

RC walls with code minimum vertical reinforcement are common when the dimensions of the wall are larger than that required for strength, or when axial loads provide sufficient flexural capacity. During the 2010/2011 Canterbury earthquakes, several lightly reinforced concrete walls in multi-storey buildings formed a limited number of cracks in the plastic hinge region as opposed to the expected well distributed cracking (Kam et al. 2011; Structural Engineering Society of New Zealand (SESOC) 2011; Bull 2012). This behaviour was also observed in the RC walls of the EI Faro building during the 1985 Chilean Earthquake (Wood 1989; Wood et al. 1991).

The unexpected failure modes of several lightly RC walls during the Canterbury Earthquakes cast doubts on whether RC walls designed according to current code specified minimum vertical reinforcement requirements can support the formation of distributed cracks and exhibit sufficient ductility and deformation capacity during earthquakes. In the current version of the NZ Concrete Structures Standard, NZS 3101:2006, the minimum reinforcement equation for walls is simply the minimum longitudinal reinforcement equation developed for RC beams. This equation was initially intended to ensure that there is sufficient separation between the nominal flexural strength and the cracking strength of RC beams. However, typical wall and beam details are distinctly different and may result in a reduced safety margin between the flexural and cracking strength of typical wall sections. The results of moment-curvature analyses performed by Henry (2013) indicated that RC walls with minimum reinforcement in accordance with NZS 3101:2006 may be susceptible to brittle failure unless a significant axial load was applied. In addition, despite a significant number of RC walls tests having been conducted over the last three decades, there is a lack of experimental testing or modelling of lightly reinforced walls that are designed according to code specified minimum reinforcement requirements. A systematic evaluating of current minimum vertical reinforcement limits for walls is imperative.

An experimental program is described that was designed to evaluate the minimum vertical reinforcement limits in the NZS 3101:2006. Additionally, a series of numerical analyses were conducted to investigate the behaviour of the test walls and a lightly RC wall in the Gallery Apartments building that was damaged during the Canterbury earthquakes.

2 EXPERIMENT INTRODUCTION

2.1 Specimen design

A series of experimental tests of reinforced RC walls designed in accordance with current minimum vertical reinforcement limits in NZS 3101:2006 have been planned. The wall test specimens were designed to approximately represent a 40-60% scale version of multi-storey RC walls with limited ductility. Vertical reinforcement was designed in accordance with Eq. (1) from NZS 3101:2006.

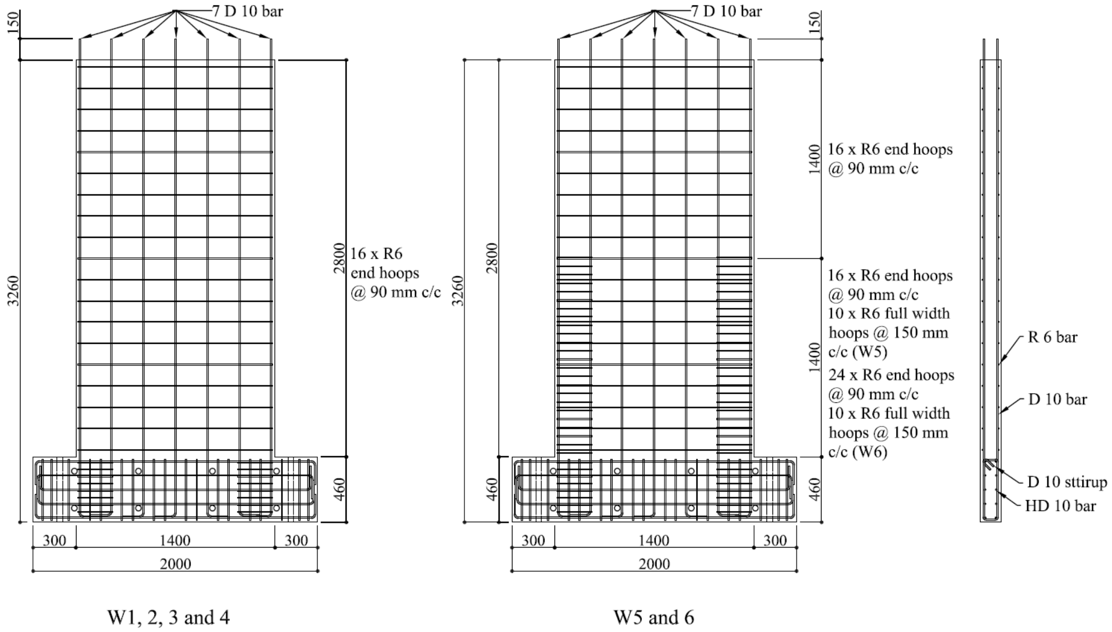
$$\rho_{\min} = \frac{\sqrt{f_c^{'}}}{4 f_y} \tag{1}$$

All six walls had the same material properties with a concrete compressive strength of 40 MPa and vertical reinforcement yield strength of 300 MPa. Using Eq. (1), these material properties correspond to a minimum vertical reinforcement ratio of 0.53%. The walls all had the same dimensions and reinforcement, with variations in either the aspect ratio or axial load ratio considered. Three aspect ratios will be applied to the test walls, 2, 4, and 6, representing walls in low to high buildings. The applied axial load will also be varied from 0-10% of the wall's axial capacity. A summary of the details for each of the six test walls is shown in Table 1.

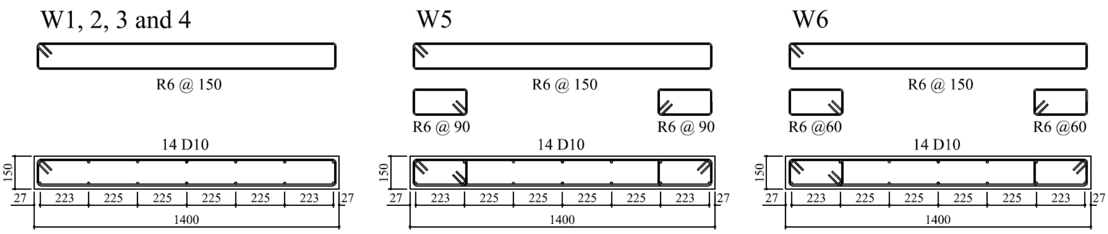
Table 1. Details of the first series of RC wall tests

Specimen	Aspect ratio (M/VL_w)	Axial load ratio	Materials		Vertical reinforcement ratio (%)
			$f_c^{'}$ (MPa)	f_y (MPa)	
W1	2	3.5%	40	300	0.53
W2	4	3.5%	40	300	0.53
W3	6	3.5%	40	300	0.53
W4	4	0	40	300	0.53
W5	4	7%	40	300	0.53
W6	4	10%	40	300	0.53

As shown in Figure 1, the wall test specimens have a length of 1.4 m, a thickness of 150 mm, and a height of 2.8 m. The height represents the lower two storeys of the prototype walls and is equal to twice the wall length to ensure that the expected region of inelastic behaviour is included within the specimen. Wall 5 and wall 6 required additional confinement reinforcement in the end regions to achieve a limited ductile response as stated in NZS 3101:2006.



(a) Elevation



(b) Cross sections

Figure 1. Details of test wall specimens

2.2 **Test setup**

Because of the height limitation of the laboratory, a test setup was designed to simulate the expected seismic loading on the bottom two storeys of a 40-60% scaled wall. Based on an assumed lateral-load distribution, the moment, shear, and axial loads at the second storey height can be calculated, as shown in Figure 2.

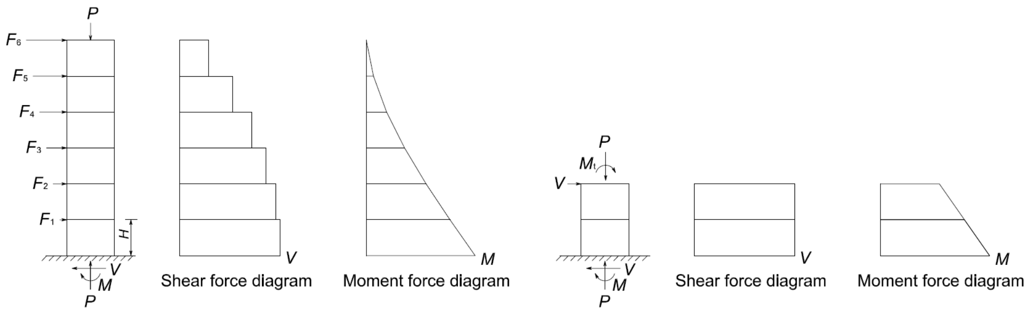


Figure 2. Seismic loading on multi-storey RC walls

The test setup developed for the RC wall specimen is shown in Figure 3. An actuator is attached between the steel loading beam and the laboratory strong wall to apply horizontal loads to the wall, and two actuators are attached vertically at each end of the wall to achieve the required moment and axial load at the top of the wall. For high axial load cases, the capacity of the vertical actuators may be exceeded and additional axial load will be provided by external post-tensioned bars.

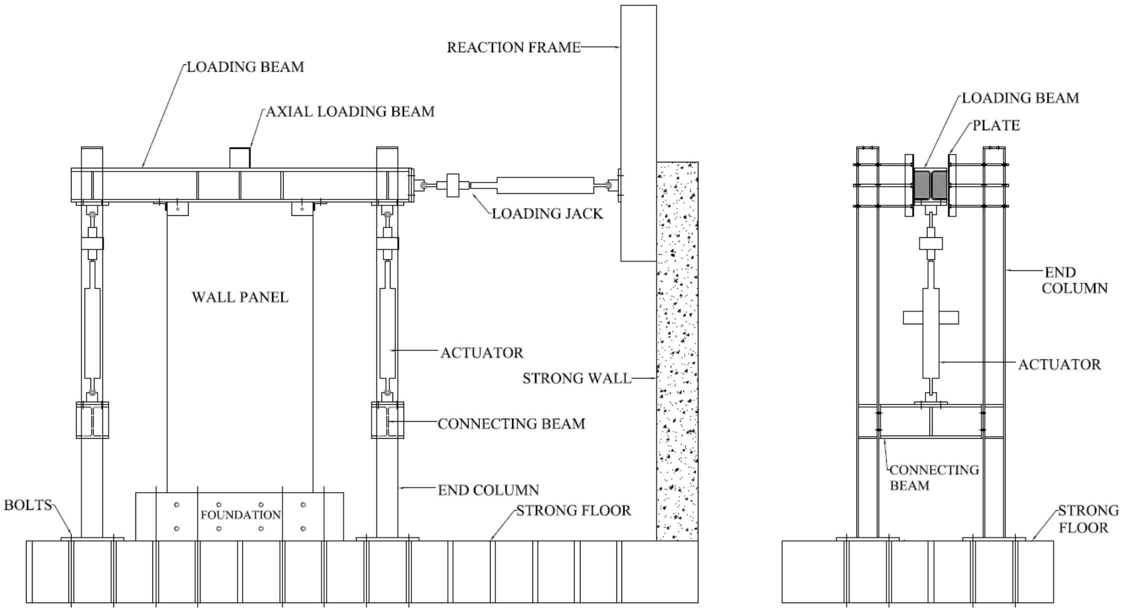


Figure 3. Experimental test setup for RC walls

The testing of the walls is currently in progress and results are not yet available at the time of publication. The results of the first series of tests will be used to determine the stiffness, extent of crack distribution, and drift capacity of walls with code specified minimum vertical reinforcement. Following these tests, a second series of walls will be tested with the aim of improving the performance from the first series and refining any potential modification to minimum vertical reinforcement limits.

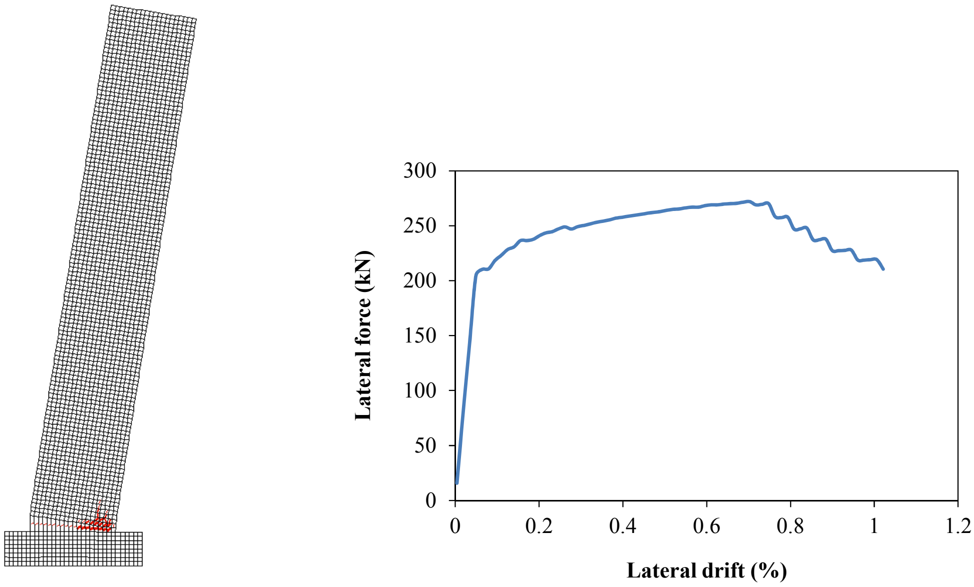
3 NUMERICAL MODELLING

Prior to the experimental tests, a series of numerical analyses were conducted to investigate the lateral load response of the six test walls and the Grid-F wall from the Gallery Apartments building in Christchurch. The models was conducted using nonlinear finite element program VecTor2 (Wong and Vecchio 2003), which has been successfully implemented for modelling RC walls(Ghorbani-Renani et al. 2009; Luu et al. 2013). Four–node plane stress rectangular elements were used to model the RC walls with smeared horizontal reinforcement and truss elements were used to model the vertical reinforcements. The axial compression due to gravity loads was held constant during the analyses, whereas the lateral load applied at the top of the wall was monotonically increased until failure. The constitutive law for concrete in compression used the Hognestad parabolic model, with a Park-Kent (Park et al. 1982) descending branch. The fib model code recommendation was adopted for the uniaxial tensile strength of the concrete (Fédération Internationale du Béton (fib) 2012) and a trilinear stress-strain response was used for the reinforcement. Detailed descriptions of the material models can be found in the VecTor2 user manual (Wong and Vecchio 2003).

3.1 Analysis of Grid-F wall in the Gallery Apartments building

The Gallery Apartments building damaged during the Canterbury earthquakes was designed according to the 1995 version of the NZ Concrete Structures Standard, NZS 3101 (Smith and England 2012). The grid-F wall had a length of 4300 mm and a thickness of 325 mm. The vertical reinforcement ratio was 0.16%, greater than the 0.14% minimum limit required by NZS 3101:1995. The wall was 39 m high corresponding to an aspect ratio of 6.1 when assuming an inverse triangular lateral force distribution. The average measured concrete strength was 51.3 MPa with a corresponding tensile strength of 4.34 MPa. The vertical reinforcement had a yield strength of 560 MPa, an ultimate strength of 690 MPa and an ultimate strain of 12.9%. The axial load acting on the grid-F wall was 2250 kN, corresponding to an axial load ratio of 3.0%.

Figure 4 shows the crack pattern and load-drift curve of the modelled Grid-F wall from Gallery Apartments. The behaviour of the modelled as-built grid-F wall was similar to the failure mode observed during the 22 Feb 2011 Christchurch earthquake, with a single crack at the wall base. The wall demonstrated only limited ductility with fracture of vertical reinforcement occurring at only 0.75% lateral drift, as shown in Figure 4b.



(a) Predicted crack pattern **(b) Lateral force-drift response**

Figure 4. Modelled behaviour of grid-F wall in Gallery Apartments building

3.2 Analysis of test walls

Additional push-over analyses were conducted for the six test walls described previously. The walls all had vertical reinforcement consistent with current minimum limits in NZS 3101:2006 and the section and reinforcement details were shown in Table 1 and Figure 1. The concrete strength used in the model was defined as the specified 28-day concrete strength of 40 MPa, with a corresponding tensile strength of 3.51 MPa. The top beam was modelled as a stiff concrete beam. The reinforcement had a yield strength of 300 MPa, an ultimate strength of 409 MPa and an ultimate strain of 15.0%.

Figure 5 shows the loading condition applied to the wall models to simulate the test loading conditions. The axial load was kept constant throughout the analysis with half applied at each end of the top beam. The horizontal load was applied in the center of the beam and increased monotonically throughout the analysis. To model the walls with aspect ratio 4 and 6, the required moment at the top of the wall was applied by a pair of vertical loads at each end of the top beam. To keep the aspect ratio constant, these vertical loads were maintained at a constant ratio to the horizontal load at each step, as shown by Eq. (2a) and Eq. (2b). The variables l_w, λ, H_e and L are all known and so the vertical loads were equal to 1.83 times and 3.77 times of the horizontal load for walls with aspect ratio 4 and 6, respectively. For the wall with aspect ratio 2, the vertical loads required equalled to zero.

$$\lambda = \frac{F_h H_e + F_v L}{F_h l_w} \tag{2a}$$

$$F_{v1} = -F_{v2} = -\frac{F_h l_w \lambda - F_h H_e}{L} = -\frac{l_w \lambda - H_e}{L} F_h \tag{2b}$$

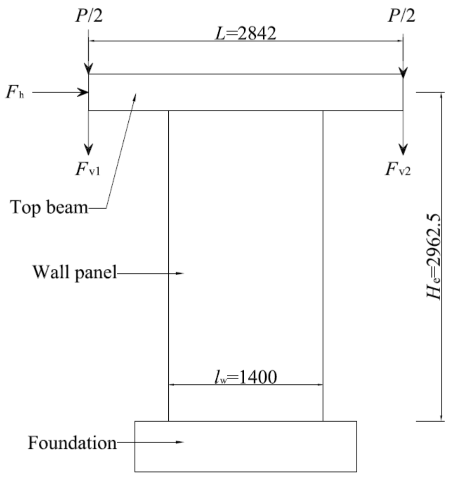
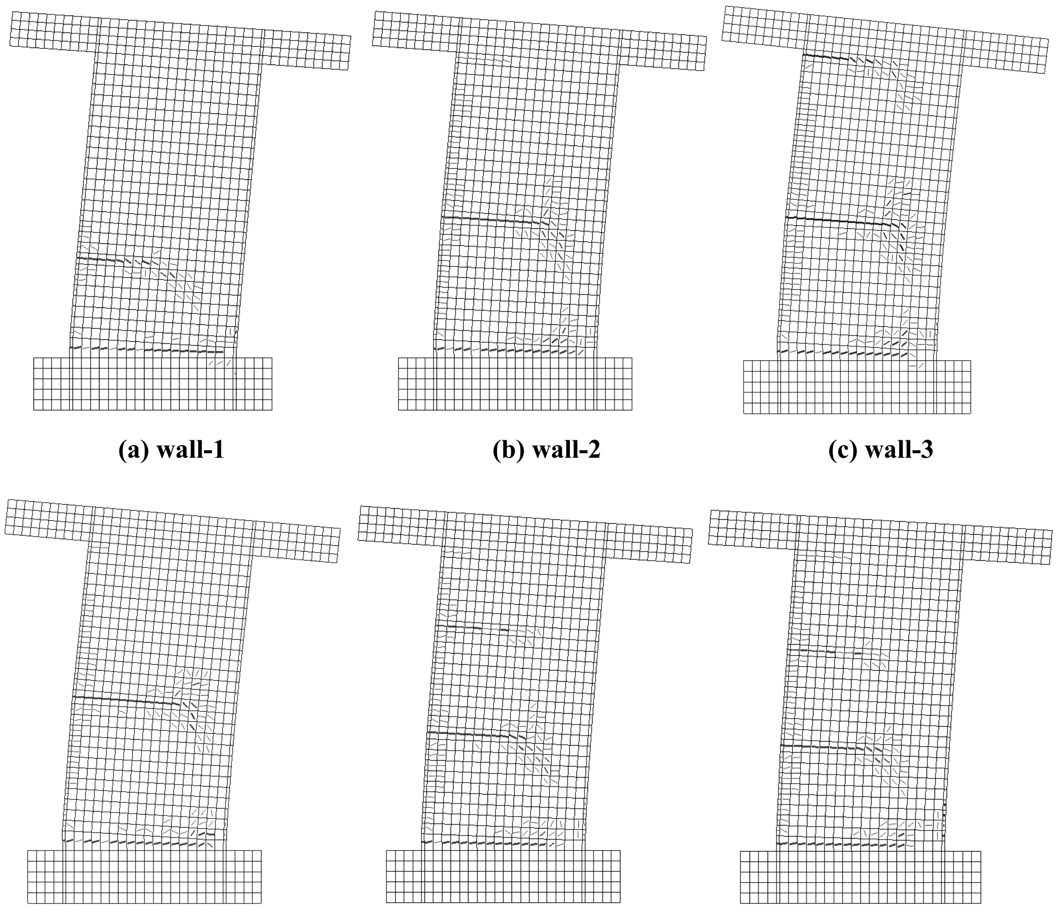


Figure 5. Loading condition applied to the test wall model (dimensions in mm)

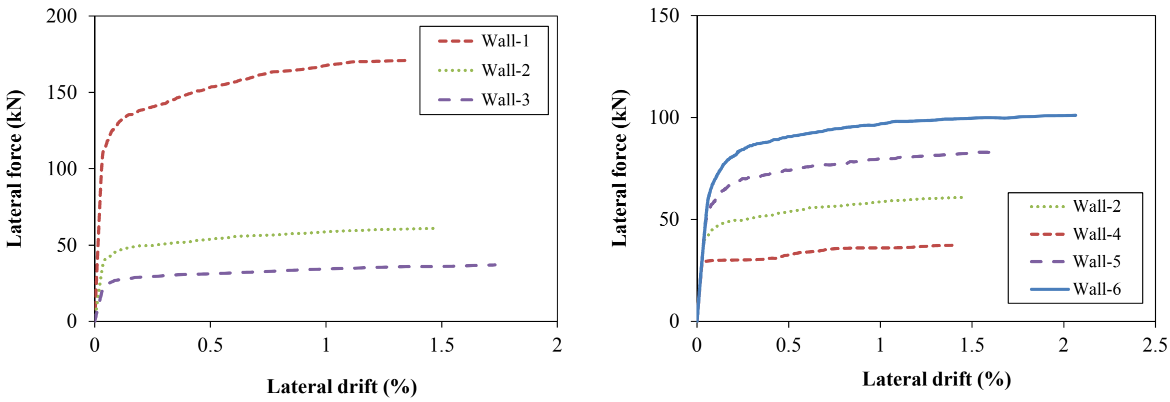
Figure 6 and Figure 7 show the crack patterns and lateral force-drift response calculated for each of the test walls modelled. As shown in Figure 6, all the six walls formed two to three primary flexural cracks with some secondary cracks at the edge that did not propagate along the wall length. All six analyses were terminated when fracture of the vertical reinforcement occurred at lateral drifts of around 1.5%. The walls designed in accordance with current design standards showed an improved lateral-load response when compared to the as-built grid-F wall which had a drift capacity of only 0.75%. However, the calculated displacement capacity for all walls was still less than the allowable drift limits for ductile buildings.



(a) wall-1 **(b) wall-2** **(c) wall-3**

(d) wall-4 **(e) wall-5** **(f) wall-6**

Figure 6. Deformed shape (magnified x5) and crack patterns of test walls modelled



(a) Wall-1, 2 and 3

(b) Wall-2, 4, 5 and 6

Figure 7. Calculated lateral force-drift response for test walls modelled

Wall-1, 2 and 3 are identical except for the aspect ratio that was varied from 2-6. As the aspect ratio was increased, a greater number of primary and secondary cracks were observed in the wall. For wall-1 with an aspect ratio 2, two primary cracks formed in the lower part of the wall and the deformation capacity was predominantly attributed to the first crack at the wall base. For the higher aspect ratio wall-2 and 3, three primary cracks and a greater number of secondary cracks were observed due to the more evenly distributed moments up the height of the wall. The greater distribution of cracks led to an increase in the lateral drift capacity prior to fracture of the vertical reinforcement as the aspect ratio was increased, as highlighted in Figure 7a.

Wall-2, 4, 5 and 6 are modelled to investigate the influence of axial load. As shown in Figure 6 and Figure 7b, an increase in axial load also resulted in a greater distribution of primary and secondary cracks and a larger drift capacity. From Figure 7, the drift capacity of walls with axial load ratio 0, 3.5%, 7% and 10% were 1.41%, 1.47%, 1.64% and 1.76%, respectively. It is interesting to note that these results contradict the results from previous research. Greifenhagen (2005) concluded that when the axial force ratio increasing from 0.025 to 0.1, the drift capacity of tested walls decreasing from 2.15% to 1.25%. Also, for the walls tested by Su (2007), it was indicated that the ductility was reduced from 3.05 (for W1) to 2.25 (for W2) when the axial load ratio increased from 0.25 to 0.5. An inverse trend between axial load and drift capacity is understandable for walls that are compression controlled, however, the failure of the walls modelled are controlled by fracture of vertical reinforcement. In the case of lightly reinforced concrete walls, the axial load ratio has positive effect on the drift capacity of the wall. This finding is consistent with recent research by Henry (2012) which concluded that the margin of safety between the cracking moment and flexural strength for lightly reinforced walls improved as the axial load acting on the wall was increased.

4 CONCLUSION

An investigation is currently being conducted to evaluate the optimum code minimum vertical reinforcement limits for RC walls. A series of experimental tests are planned for RC walls designed in accordance with NZS 3101:2006. These tests will help to evaluate the seismic performance of lightly reinforced RC walls meeting current minimum code requirement.

A series of numerical analyses were conducted using nonlinear finite element program VecTor2. Based on the analysis results of the modelled walls, the following conclusions were drawn:

1. The model of the grid-F wall in the Gallery Apartments building wall correctly modelled the failure mode observed during the 22 Feb 2011 Christchurch earthquake, with a single crack at the wall base and fracture of the vertical reinforcement at a lateral drift of 0.75%.
2. The test walls designed in accordance with current minimum vertical reinforcement requirements in NZS 3101:2006 may be susceptible to limited flexural cracking and premature

fracture of vertical reinforcement.

3. The drift capacity of concrete walls with code specified minimum vertical reinforcement improves as the aspect ratio and axial load ratio are increased.

5 ACKNOWLEDGMENTS

Financial support for this research was provided by the Natural Hazards Research Platform through contract C05X0907 in addition to the Chinese Scholarship Council.

REFERENCES

Bull, D. 2012. The performance of concrete structures in the Canterbury earthquakes: Lessons for concrete buildings. *Structural Engineers Association of California (SEAOC) Convention*, Santa Fe, NM.

Fédération Internationale du Béton (fib). 2012. Model Code 2010 - Final draft, Volume 1. fib Bulletin, 65. Lausanne, Switzerland.

Ghorbani-Renani, I., N. Velev, et al. 2009. "Modeling and testing influence of scaling effects on inelastic response of shear walls." *ACI Structural Journal*, Vol **106**(3): 358-367.

Greifenhagen, C. and P. Lestuzzi. 2005. "Static cyclic tests on lightly reinforced concrete shear walls." *Engineering Structures*, Vol **27**(11): 1703-1712.

Henry, R. S. 2013. "Assessment of minimum vertical reinforcement limits for RC walls." *Bulletin of the New Zealand Society for Earthquake Engineering*, Vol **46**(2): 88-96.

Kam, W. Y., S. Pampanin, et al. 2011. "Seismic performance of reinforced concrete buildings in the 22 Feburary Christchurch (Lyttelton) earthquake." *Bulletin of the New Zealand Society for Earthquake Engineering*, Vol **44**(4): 239-278.

Luu, H., I. Ghorbanirenani, et al. 2013. "Numerical modeling of slender reinforced concrete shear wall shaking table tests under high-frequency ground motions." *Journal of Earthquake Engineering*, Vol **17**(4): 517-542.

NZS 3101:1995. 1995. Concrete Structures Standard. Wellington, New Zealand, *Standards New Zealand*.

NZS 3101:2006. 2006. Concrete Structures Standard. Wellington, New Zealand, *Standards New Zealand*.

Park, R., M. J. N. Priestley, et al. 1982. "DUCTILITY OF SQUARE-CONFINED CONCRETE COLUMNS." *ASCE J Struct Div*, Vol **108**(ST4): 929-950.

Smith, P. and England, V. 2012. Independent assessment on earthquake performance of Gallery Apartments - 62 Gloucester Street. http://canterbury.royalcommission.govt.nz/documents-by-key/20120217.3188, *Report prepared for the Canturbury Earthquakes Royal Commission*.

Structural Engineering Society of New Zealand (SESOC). 2011. Preliminary observations from Christchurch earthquakes. http://canterbury.royalcommission.govt.nz/documents-by-key/20111205.1533, *Report prepared for the Canturbury Earthquakes Royal Commission*.

Su, R. K. L. and S. M. Wong. 2007. "Seismic behaviour of slender reinforced concrete shear walls under high axial load ratio." *Engineering Structures*, Vol **29**(8): 1957-1965.

Wong, S. M. and F. J. Vecchio. 2003. VecTor2 and formworks user's manual. . Toronto (Canada), *University of Toronto*.

Wood, S. L. 1989. "Minimum tensile reinforcement requirements in walls." *ACI Structural Journal*, Vol **86**(5): 582-591.

Wood, S. L., R. Stark, et al. 1991. "Collapse of eight-story RC building during 1985 Chile earthquake." *Journal of Structural Engineering*, Vol **117**(2): 600-619.