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RESEARCH TO ADDRESS THE PERFORMANCE OF LIGHTLY REINFORCED CONCRETE WALLS DURING THE 2010/2011 CANTERBURY EARTHQUAKES

Richard S. HENRY[1] and Yiqiu LU[2]

ABSTRACT

During the 2010/2011 Canterbury earthquakes, several reinforced concrete (RC) walls in multi-storey buildings formed a limited number of cracks at the wall base with fracture of vertical reinforcement occurring. This failure mode is typical of lightly reinforced concrete members, where the area of reinforcing steel is insufficient to develop the tension force required to form secondary cracks. The minimum vertical reinforcement limits for RC walls in different concrete design standards were compared, and a series of numerical analyses were used to investigate the behaviour of an example RC wall designed according to the minimum requirements of each standard. The analysis results confirmed the observed failure mode of an RC wall with less than the current minimum vertical reinforcement that was damaged during the Canterbury earthquakes. Furthermore, RC walls built in accordance with current minimum vertical reinforcement requirements in both ACI 318-11 and NZS 3101:2006 are still susceptible to limited flexural cracking and premature bar fracture. The ductility of RC walls with concentrated reinforcement at the wall ends, such as that required by Eurocode 8, CSA 2004 and GB 50010, was significantly improved. A comprehensive experimental program is also currently underway to verify the seismic performance of lightly reinforced concrete walls, and a test setup has been developed to subject RC wall specimen to loading that is representative of a multi-storey building. Numerical analyses of the test walls further confirmed that RC walls designed according to current NZS 3101:2006 minimum vertical reinforcement requirements may be susceptible to limited flexural cracking and premature reinforcement fracture. Additionally, the analyses indicated that drift capacity of RC walls with minimum vertical reinforcement improved as the aspect ratio or axial load ratio was increased.

INTRODUCTION

The 2010/2011 Canterbury earthquake series in New Zealand tested the built infrastructure to beyond the design level seismic loading and caused significant damage to both traditional and modern reinforced concrete buildings. In particular, severe damage was observed to reinforced concrete (RC) walls in several modern multi-storey buildings. As reported by Kam et al. (2011), approximately 30% of modern RC wall buildings in the Christchurch CBD were tagged as unsafe immediately following the 22 Feb 2011 earthquake, and two buildings with RC walls collapsed. Undesirable failure modes that were observed included a lack of distributed flexural cracks, premature fracture of vertical reinforcement, global and local wall buckling, bar buckling, shear failure, failure of reinforcement splices, and evidence of poor detailing. A comprehensive description of the performance of RC walls during the Canterbury earthquakes and analysis of common vulnerabilities has been published by

[1] Lecturer, Univeristy of Auckland, Auckland, New Zealand, rs.henry@auckland.ac.nz
[2] PhD student, Univeristy of Auckland, Auckland, New Zealand, ylu765@aucklanduni.ac.nz

Sritharan et al. (2014). Additionally, the Canterbury Earthquakes Royal Commission (CERC) highlighted the need for further research to improve the seismic design of RC walls and there have been interim design recommendations developed by the Structural Engineering Society of New Zealand (SESOC) (Structural Engineering Society of New Zealand (SESOC) 2011; Canterbury Earthquakes Royal Commission 2012).

During an earthquake, ductile cantilever RC walls are designed to form a plastic hinge at the base of the wall. An example of well distributed flexural cracking at the base of an RC wall in a building that performed well during the Canterbury earthquakes is shown in Figure 1a. However, several other RC walls in multi-storey buildings formed a limited number of cracks in the plastic hinge region (Kam et al. 2011; Sritharan et al. 2014). Two examples of RC walls that formed only limited flexural cracks during the Canterbury earthquakes are shown in Figure 1b and Figure 1c. After breaking out the surrounding concrete it was found that the vertical reinforcing steel was often fractured due to the inelastic demand at the crack location, as shown in Figure 1d. This type of failure is characteristic of RC members with low vertical reinforcement contents and was also observed in buildings following the 1985 Chilean Earthquake (Wood 1989; Wood et al. 1991). If too little vertical reinforcement is used in walls there can be a sudden loss of strength and stiffness when the wall first cracks leading to large displacement demands and fracture of vertical reinforcement. Additionally, tension generated in the reinforcement at the first crack may be insufficient to generate secondary cracking, resulting in a reduced number of cracks, large crack widths, and possible fracture of the vertical reinforcement. The Canterbury Earthquake Royal Commission recommended that research be conducted to refine design requirements for crack control in RC walls (Canterbury Earthquakes Royal Commission 2012). A summary of current research underway to address this recommendation is presented. A review of historic and current minimum vertical reinforcement requirements was conducted, followed by extensive numerical modelling and experimental testing of lightly reinforced concrete walls.

(a) Well distributed flexural cracks (Kam et al. 2011) (b) Few flexural cracks (Credit: Charles Clifton) (c) Few flexural cracks (Credit: Ken Elwood) (d) Fractured bars (Credit: Des Bull)

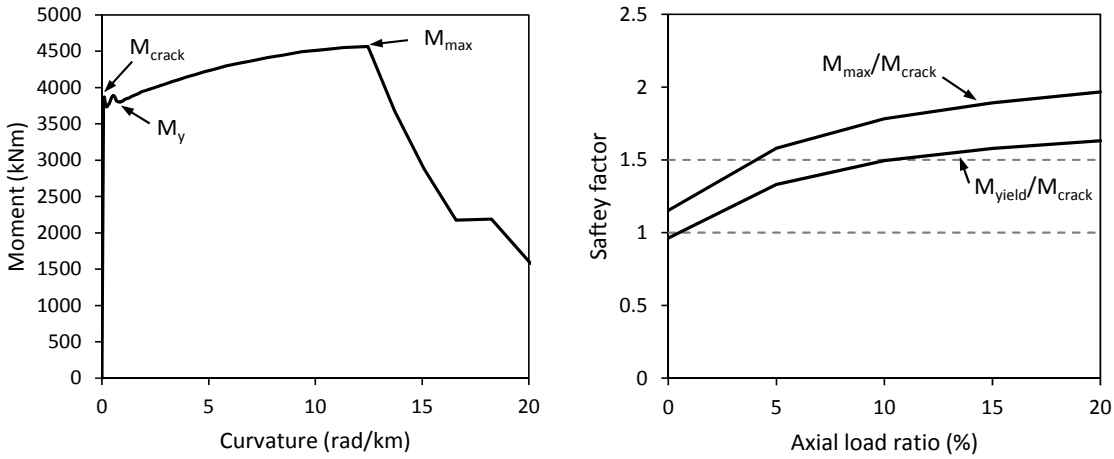
Figure 1. Examples of observed damage to lightly reinforced concrete walls in Christchurch buildings

MINIMUM VERTICAL REINFORCEMENT LIMITS

Historically, minimum requirements for vertical reinforcement in RC walls were governed by shrinkage and temperature effects, resulting in reinforcement contents as low as 0.14%. More recently, minimum vertical reinforcement limits for RC walls have been increased in design standards worldwide to ensure that ductile behaviour is achieved when yielding of reinforcement is expected. In the 2006 revision of the New Zealand Concrete Structures Standard, NZS 3101 (2006), the minimum required vertical reinforcement in RC walls was increased by over 80% with the adoption of Eq. 1, where ρ_n is the ratio of vertical reinforcement area to the gross section area (A_t/A_g).

$$\rho_n \geq \frac{\sqrt{f_c'}}{4 f_y} \tag{1}$$

Eq. 1 was adopted from a similar equation specified for RC beams that was originally developed to ensure that the nominal flexural strength was more than 1.5 times greater than the probable cracking moment of a rectangular beam (ACI 363R-84 1984). However, there are several obvious differences between RC beams and walls that make the suitability of Eq. 1 questionable (Henry 2013). A series of moment-curvature analyses were conducted to investigate the sectional response of a typical RC wall with minimum vertical reinforcement in accordance with Eq. 1. As shown in Figure 2a, the moment-curvature response of lightly reinforced wall with no axial load is dominated by a high initial stiffness and high cracking moment close to the strength at first yield of the reinforcement. From this analysis it was concluded that the current minimum vertical reinforcement limits in NZS 3101:2006 may be insufficient to ensure well distributed cracking in ductile plastic hinge regions (Henry 2013). Additionally, an axial load ratio greater than 5% may be required to ensure that the flexural strength is greater than the probable cracking strength of the wall, as is highlighted in Figure 2b.



(a) Moment-curvature of wall with no axial load (b) Effect of axial load

Figure 2. Sectional analysis results for an RC wall designed with NZS 3101 minimum vertical reinforcement (Henry 2013)

The minimum vertical reinforcement requirements from five different concrete standards were compared, including the New Zealand concrete structures standard NZS 3101:2006 (2006), the US building code requirements for structural concrete ACI 318-11 (2011), the European seismic design standard Eurocode 8 (CEN 2004), the Canadian concrete structural design standard CSA A23.3 standard (2004), and the Chinese concrete design standard GB 50010 (2010). The total or distributed minimum required vertical reinforcement contents and additional limits on reinforcement in the end regions (boundary elements) for ductile RC walls are compared in Table 1 for each standard. Several of the standards require the total or distributed minimum vertical reinforcement to be greater than 0.25% with the limit of Eurocode 8 set slightly lower at 0.2%. The NZS minimum vertical reinforcement limit is calculated based on the concrete and reinforcing steel strengths. For a 30 MPa concrete, the NZS 3101:2006 equation (Eq. 1) will result in vertical reinforcement contents greater than either 0.27% or 0.46% depending on the reinforcing steel grade.

Table 1. Minimum vertical reinforcement requirements in different design standards for ductile RC walls

Standards	Total/distributed reinforcement ratio*	Boundary element reinforcement ratio
ACI 308-11	>0.25%	No requirement
NZS 3101: 2006	$> \sqrt{f_c'}/4f_y$	No requirement
Eurocode 8	>0.2%	>0.5%
CSA 2004	>0.25%	$>(0.15\% b_w l_w)/(b_w l_b)$
GB 50010-2010	>0.25%	>1.0%

* For NZS the minimum total vertical reinforcement ratio is specified whereas the minimum distributed reinforcement ratio in the web region of the wall is used by all other standards.

ACI 318 and NZS 3101 currently have no requirement for additional vertical reinforcement to be placed in boundary elements at the ends of the walls. The CSA and GB standards state that concentrated vertical reinforcement shall be provided at each ends of walls designed for all classes of ductility. For ductile walls, CSA states that a minimum of four bars should be placed in at least two layers in the boundary elements and that the minimum area of vertical reinforcement in the wall boundary elements in plastic hinge regions shall be at least $0.0015b_w l_w$. In the GB code the minimum vertical reinforcement content required in the boundary elements is dependent on the seismic design level, but at least greater than 1.0%. Eurocode 8 requires vertical reinforcement content of at least 0.5% in the boundary region for ductile walls.

NUMERICAL MODELLING

A series of numerical analyses were conducted to investigate the lateral load response of lightly reinforced RC walls using nonlinear finite element program VecTor2 (Wong and Vecchio 2003). Several researches have previously validated the accuracy of VecTor2 for modelling the lateral load behaviour of RC walls (Ghorbani-Renani et al. 2009; Luu et al. 2013). In this study, four–node plane stress rectangular elements were used to model the RC walls with smeared horizontal and vertical reinforcement. 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 in a displacement-control mode until failure. The constitutive law for concrete in compression uses the Hognestad parabola 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 tri-linear 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).

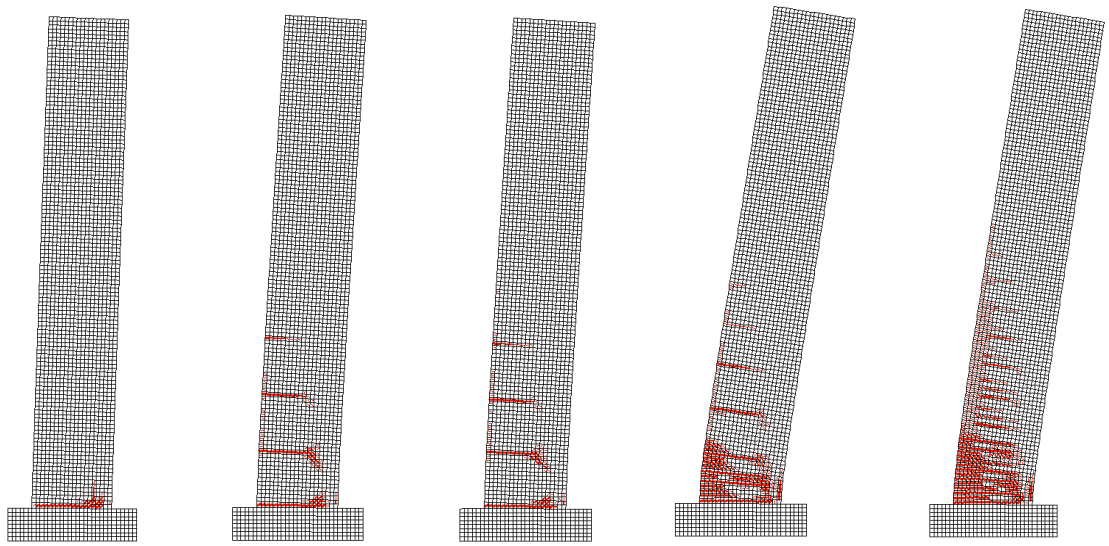
The grid-F wall from the Gallery Apartments building in Christchurch was used as the baseline for the VecTor2 analyses. The grid-F wall had a length of 4300 mm, a thickness of 325 mm, two layers of DH12 vertical reinforcement at 460 mm centers and DH12 horizontal reinforcement at 400 mm centers. The vertical reinforcement ratio for the as-built grid-F wall was 0.16%, less than the 0.27% currently required by NZS 3101:2006. The Gallery Apartments building was constructed prior to the adoption of the current NZS 3101:2006 version when the minimum vertical reinforcement was only required to exceed 0.14%. The grid-F wall was 39 m high corresponding to shear span ratio of 6.1 when using an inverse triangular lateral force distribution. The specified 28-day concrete strength of the RC walls was 30 MPa, however, tests performed on two concrete cores extracted from the building indicated compressive strengths of 46.5 and 56.0 MPa. As a result, the as-built grid-F model was analysed using the average measured concrete strength of 51.3 MPa with a corresponding tensile strength of 4.34 MPa. The reinforcement properties specified were based on previous testing of grade 500 reinforcing steel, with 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%.

Additional analyses were conducted using the dimensions of the gird-F wall with modified reinforcement detailing in accordance with the current minimum requirements from each of the standards discussed earlier. The boundary lengths of Eurocode, CAS and GB were taken as $0.15l_w$, which was equal to 645 mm. The concrete strength was defined as the specified 28-day concrete strength of 30 MPa, with a corresponding tensile strength of 2.93 MPa. The reinforcing steel properties were kept the same as that described for the as-built grid-F wall. The resulting reinforcement contents for each of the walls modelled are summarised in Table 2. The walls designed in accordance with ACI and NZS had evenly distributed vertical reinforcement, whereas the walls designed in accordance with Eurocode, CAS and GB had additional reinforcement lumped in ends (boundary elements) with distributed reinforcement along the web region.

Table 2. Summary of reinforcement details of each of the walls modelled

Standards	Boundary reinforcement ratio ρ_b	Web reinforcement ratio ρ_w	Total reinforcement ratio
As-built grid-F	-	0.16%	0.16%
ACI 308-11	-	0.254%	0.254%
NZS 3101: 2006	-	0.275%	0.275%
Eurocode 8	0.5%	0.2%	0.29%
CSA 2004	1.0%	0.254%	0.48%
GB 50010-2010	1.0%	0.254%	0.48%

Figure 3 and Figure 4 show the predicted crack patterns and lateral force-drift response calculated for each of the walls modelled. 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 flexural crack at the wall base. The strain in the vertical reinforcement was concentrated at the single crack and not distributed along a large length of the bar. Because of the reduced spread of the plasticity, the wall demonstrated only limited ductility with fracture of vertical reinforcement occurring at only 0.75% lateral drift.



a) Grid-F b) ACI c) NZS d) Eurocode e) CAS & GB

Figure 3. Deformed shape (magnified x5) and crack patterns when the first reinforcing bar fractured for each of the walls modelled

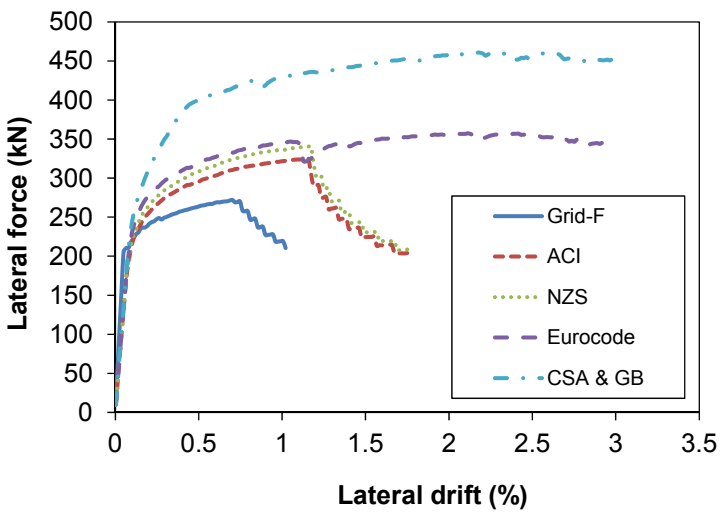


Figure 4. Calculated lateral force-drift response for each of the walls modelled

All of the walls designed in accordance with current design standards showed an improved lateral-load response when compared to the as-built grid-F wall. The crack patterns for the walls designed according to both ACI and NZS were similar with a total of four primary flexural cracks developing. The reinforcement ratio of these two walls was 58% and 72% higher than the as-built grid-F wall. The as-built wall also has a higher initial stiffness than the other walls due to the decreased number of flexural cracks. However, despite the increased number of flexural cracks in the ACI and NZS walls, the reinforcement was still insufficient to generate a large number of secondary cracks and fracture of the vertical reinforcement occurred at a modest 1.2% lateral drift. The calculated displacement capacity for the ACI and NZS walls is significantly less than the allowable drift limits for ductile buildings.

The performance of the wall designed in accordance to Eurocode was significantly better than the walls designed according to ACI and NZS. The concentrated reinforcement ratio of 0.5% in the end regions of the wall was sufficient to generate a large number of secondary cracks in the plastic hinge region, as shown in Figure 3d. However, because the total reinforcement for the Eurocode wall is not significantly larger than the NZS wall, the maximum lateral strength was similar, as shown in Figure 4. The increased secondary cracking in the Eurocode wall improved the spread of inelastic strains in the vertical reinforcement, resulting in a significantly more ductile response than that calculated for the ACI or NZS walls. The Eurocode wall indicates that concentrating a greater portion of reinforcement at the wall ends could result in a significant improvement in the seismic response of lightly reinforced concrete walls.

The performance of walls designed with CSA and GB are similar with Eurocode. Because the distributed and boundary reinforcement are both larger than Eurocode, the maximum lateral strength is higher than Eurocode. In addition, secondary cracks are denser both in the boundary elements and web region in the plastic hinge region and also distribute along a greater height of the wall. The increased lateral strength of these walls would have implications when considering member over-strengths during capacity design and the design of the foundation system.

EXPERIMENTAL TESTING

To supplement and validate the numerical modelling a series of experimental tests will be conducted. Despite an extensive number of RC walls being tested over the last three decades, few tests have been completed that represent flexure dominant walls (aspect ratio > 2) with low vertical reinforcement contents (< 1%) and low axial loads (< 10%) that are typical of many New Zealand wall designs.

The first series of experimental tests will focus on RC walls designed to examine the current minimum vertical reinforcement limits in NZS 3101:2006. A summary of the test program is shown in Table 3, and drawings of the wall specimen are shown in Figure 5. The 1.4 m long, 2.8 m high and 150 mm thick wall specimen were designed to approximately represent a 40-50% scale version of RC walls with limited ductility as per NZS 3101:2006. The vertical reinforcement was identical for all six walls and designed in accordance with minimum requirement in NZS 3101:2006 (Eq. 1). Three moment to shear (M/V) ratios equal to 2, 4, and 6 will be applied to the test walls representing walls in a range of different building heights. The applied axial load will also be varied from 0-10% of the wall axial capacity. Wall 5 and wall 6 required to additional confinement reinforcement in the ends regions to achieve a limited ductile response as per NZS 3101:2006.

Table 3. Details of the first series of RC wall tests

Specimen	Aspect ratio	Axial load ratio	Materials		Longitudinal reinforcement ratio (%)
			f_c' (MPa)	f_y (MPa)	
1	2	3.5%	40	300	0.53
2	4	3.5%	40	300	0.53
3	6	3.5%	40	300	0.53
4	4	0	40	300	0.53
5	4	7%	40	300	0.53
6	4	10%	40	300	0.53

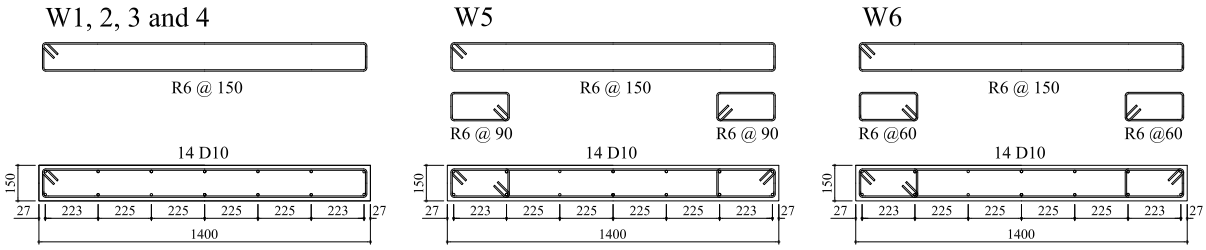


Figure 5. Cross-sections of test wall specimens

Because of the height limitation of the structural testing laboratory, a test setup was designed to simulate the expected seismic loading on the bottom two storeys of a 50% scaled wall. Based on an assumed lateral-load distribution, the moment, shear, and axial loads at the second storey height of a multi-storey building can be calculated, as shown in Figure 6. The lateral-load distribution in Figure 6 is based on a first mode response, but higher modes and/or coupling effects introduced by floor diaphragms can also be included when calculating the M/V ratio at the top of the test specimen. The test setup developed to simulate these seismic loads on the RC wall specimen is shown in Figure 7. A foundation block is post-tensioned to the laboratory strong floor and a steel loading beam is attached to the reinforcement at the top of the wall. One actuator is attached between the steel loading beam and the 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. A steel reaction frame has been designed to support the actuators as well as provide lateral restraint to the loading beam. Lateral load is applied to the specimen via displacement control in accordance with recent ACI guidelines, with the target lateral displacement history at the top of the wall comprising three cycles each to increasing maximum displacement demands. For each step, the force applied by the two vertical actuators will adjusted to achieve the target aspect ratio and axial load based on the wall displacement and lateral force being applied by the horizontal jack.

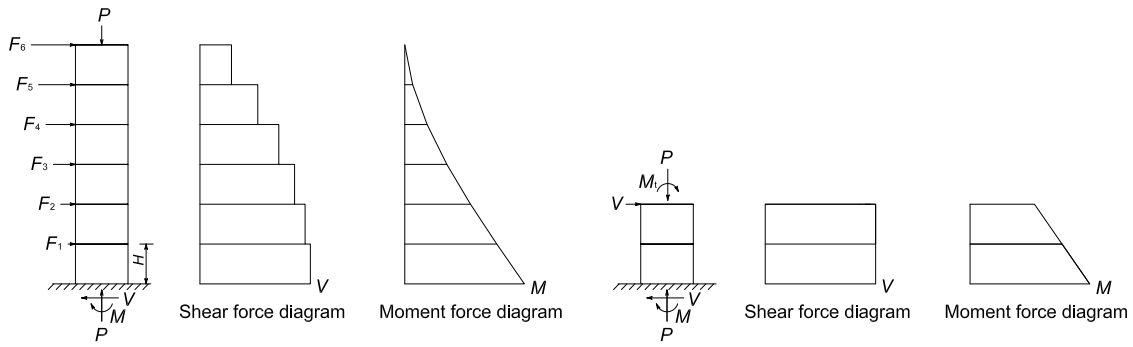


Figure 6. Seismic loading on multi-storey RC walls

MODELLING OF TEST WALLS

Prior to the experiments being completed, the six test walls were also modelled using VecTor2. The test walls were modelled in the same way as that described previously with section and reinforcement details in accordance with that shown in Table 3 and Figure 5. 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%. The loading condition applied to the modelled walls simulated 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.

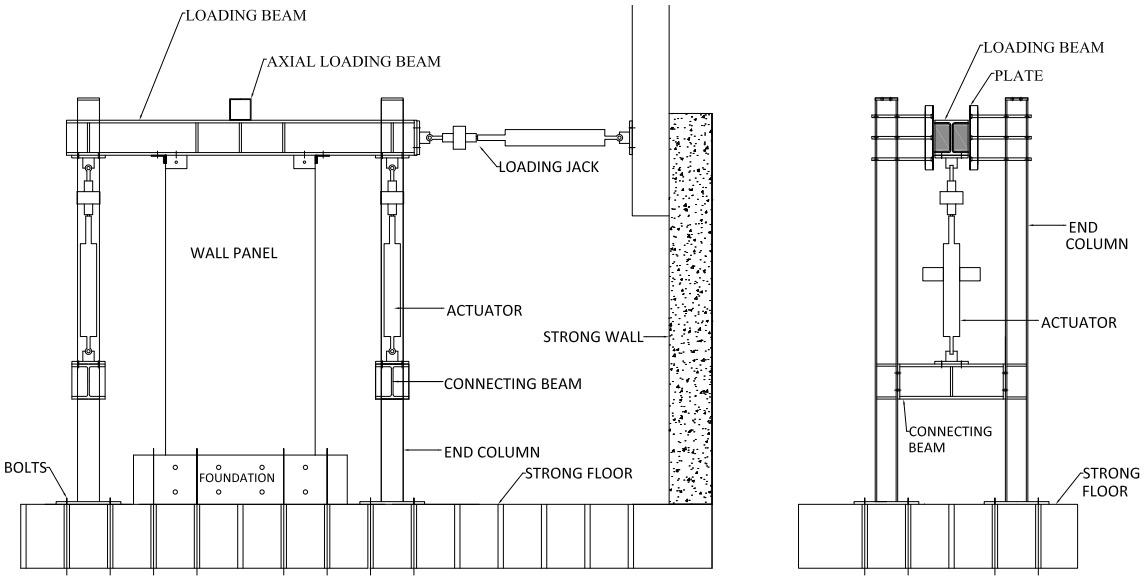


Figure 7. Experimental test setup for RC walls

Figure 8 and Figure 9 show the crack patterns and base moment-drift response calculated for each of the test walls modelled. All the six walls were predicted to form two to three primary flexural cracks with some secondary cracks at the edge that did not propagated along the wall length. All six analyses were terminated when fracture of the vertical reinforcement occurred at lateral drifts of between 1.3-1.9%. 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.

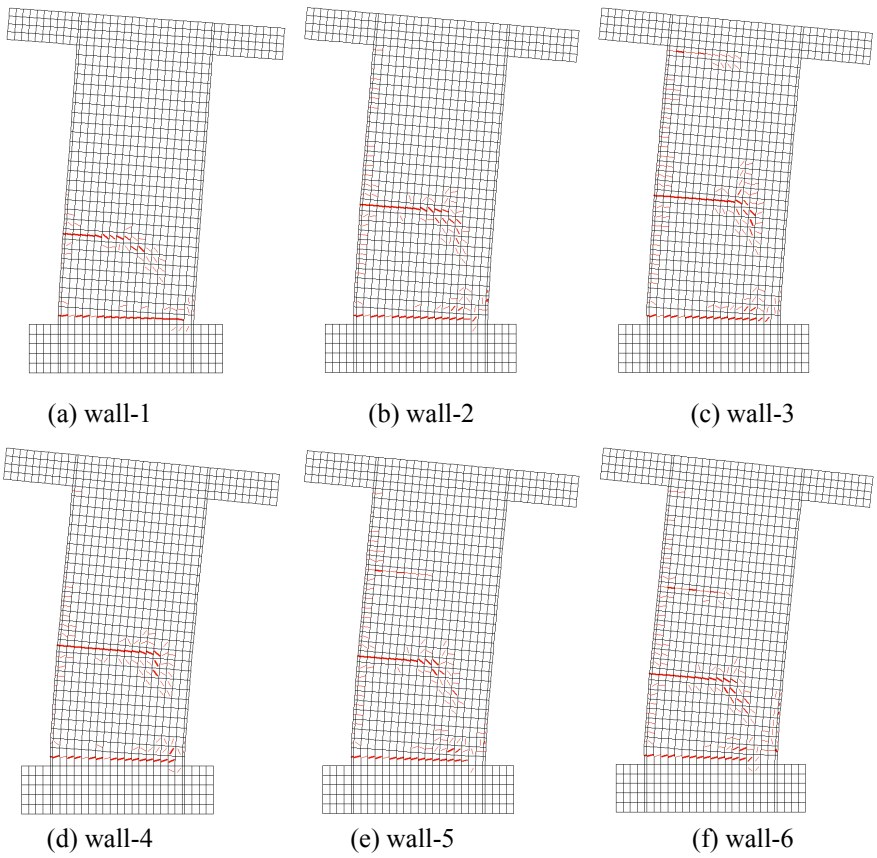
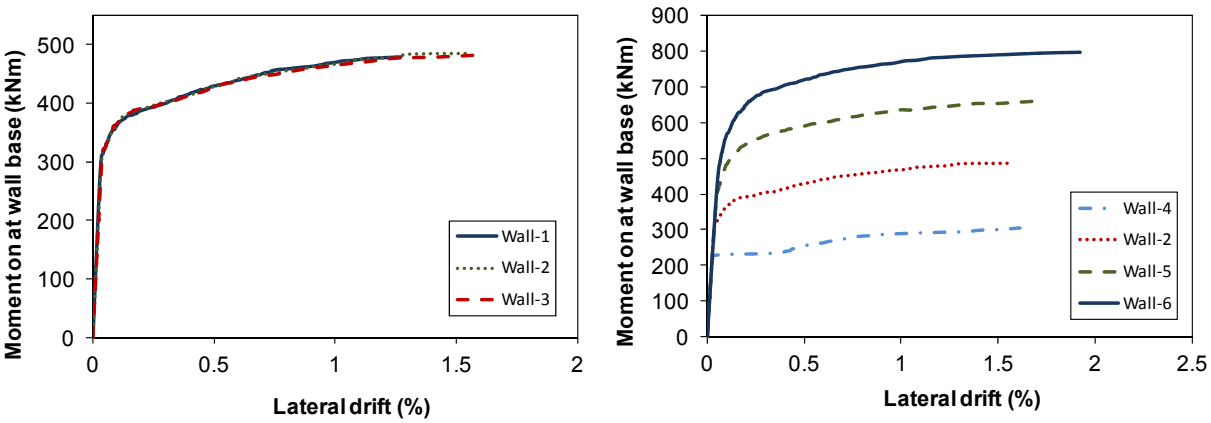


Figure 8. Deformed shape (magnified x5) and crack patterns when the first reinforcing bar fractured for each of the test walls modelled

Wall-1, 2 and 3 are identical except for the aspect ratio that was varied from 2 to 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, two and three primary cracks formed respectively and a greater number of secondary cracks were observed due to the more evenly distributed moments up the height of the wall. As shown in Figure 9a, the first layer of vertical reinforcement fractured at lateral drifts of 1.27%, 1.55% and 1.62% for walls 1, 2, and 3 respectively. These results indicate that 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.



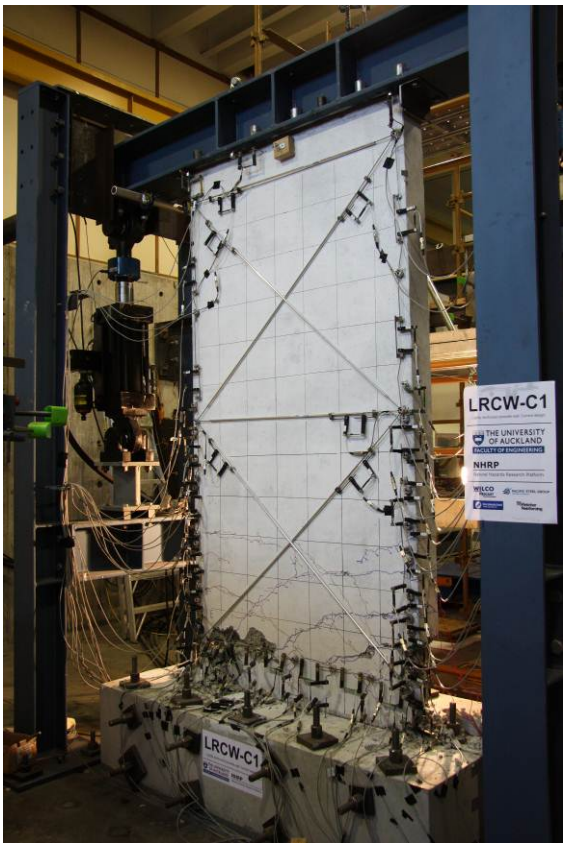
(a) Wall-1, 2 and 3 (b) Wall-2, 4, 5 and 6

Figure 9. Calculated base moment-drift response for test walls modelled

Wall-2, 4, 5 and 6 are modelled to investigate the influence of axial load. As shown in Figure 8 and Figure 9b, an increase in axial load also resulted in a greater distribution of primary and secondary cracks. All f the modelled walls failed when the vertical reinforcement was predicted to fracture at the wall base. As shown in Figure 9, the lateral drift capacity of walls with axial load ratio 0, 3.5%, 7% and 10% were 1.61%, 1.55%, 1.72%, 1.92%, respectively. It is interesting to note that these results contradict the results from previous research. For example, during RC wall tests conducted by Greifenhagen (2005) it was observed that as the axial force ratio increasing from 0.05 (for M4) to 0.1 (for M3), the drift capacity of the walls decreasing from 1.59% to 1.25%. Wall M4 failed due to a combination of concrete crushing and bar fracture, while wall M3 failed suddenly due to concrete crushing. An inverse trend between axial load and drift capacity is understandable for walls that are compression controlled, however, the failure of the walls modelled were all controlled by fracture of vertical reinforcement with no concrete crushing predicted. In the case of lightly reinforced concrete walls, the axial load ratio may have a positive effect on the drift capacity of the wall.

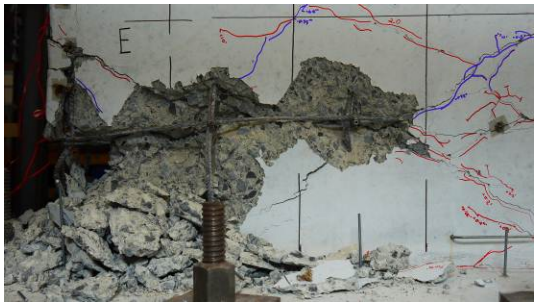
PRELIMINARY TEST RESULTS

The experimental tests are currently in progress and preliminary results for the completed wall-1 test are presented. Photos of the wall-1 condition at 2.5% lateral drift are shown in Figure 10. The wall response was dominated by flexural behaviour with 3-4 flexural cracks forming at the base of the wall. As predicted by the numerical model, the extent of flexural cracking was limited to the bottom quarter of the wall. During high lateral drift cycles, the deformation was primarily concentrated at a single wide crack with the other flexural cracks not opening wider than a few millimetres. The concrete at the corners of the wall started to spall at lateral drifts of ±1.0% and bar buckling initiated during cycles to ±1.5% lateral drift. Due to the lack of confinement reinforcement, the bar buckling accelerated concrete spalling and core crushing occurred during the first cycle to -2.5% drift. This failure mode was not predicted by the numerical analysis because reinforcement buckling was not included in the model.



(a) Overall wall condition

(b) Extent of flexural cracking



(c) Concrete crushing and bar buckling at east end

Figure 10. Photos of wall-1at 2.5% lateral drift

The measured lateral force-displacement response for wall-1 is shown in Figure 11. The uncracked wall had a high initial stiffness and the first flexural crack did not initiate until a lateral force of approximately 100 kN was reached. The inelastic response was stable up until 1.5% lateral drift when bar buckling occurred causing strength degradation on subsequent cycles. A large drop in strength occurred when the core crushed during the first cycles to -2.5% lateral drift. The strength degradation continued and two of the vertical reinforcing bars fractured on the third cycle to -2.5% lateral drift. The test was terminated after three cycles to ±2.5% lateral drift.

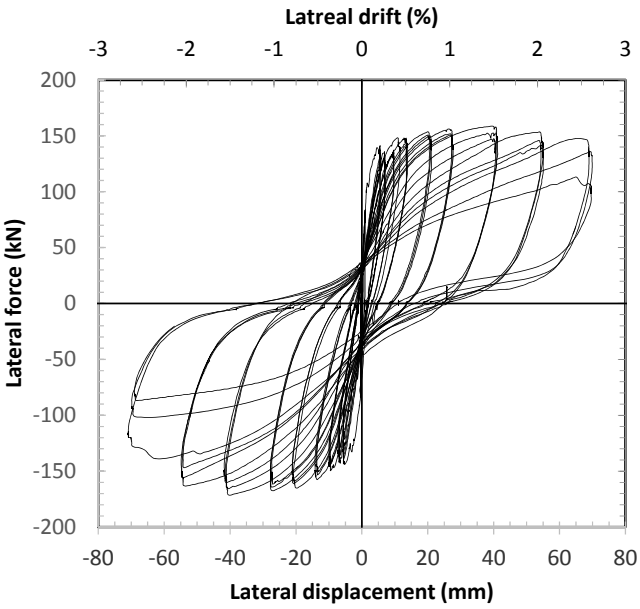


Figure 11. Lateral force-displacement response for wall-1

The preliminary results indicated that the response of wall-1 was similar to that predicted by the numerical model with limited flexural cracking at the wall base. However, the onset of bar buckling that occurred during the test was not accounted for in the model. The spalling of cover concrete and bar buckling during the test resulted in the reinforcement being stretched over a long length and possibly delaying fracture of the vertical reinforcement until higher drifts than that predicted by the model. Interestingly, the NZS 3101:2006 standard does not currently require reinforcement for lateral restraint of vertical reinforcement when the vertical reinforcement content is less than $3/f_y$ for limited ductile designs. Based on the preliminary test results, it appears that lightly reinforced walls are particularly prone to reinforcement buckling due to a small number of wide cracks that dominated the lateral load response. Future tests will repeat test wall-1 but with the inclusion of additional stirrups to restrain the vertical reinforcement from buckling. Additionally, the reinforcement buckling will be incorporated into refined numerical models.

CONCLUSIONS

An investigation is currently being conducted to evaluate the optimum minimum vertical reinforcement limits for RC walls. Current minimum vertical reinforcement provisions in several design standards were compared and 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 requirements. Prior to the experimental tests, 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. All concrete standards have minimum distributed vertical reinforcement requirements but some require additional vertical reinforcement to be placed in the boundary elements at the ends of the walls. NZS 3101:2006 is the only standard to account for the concrete and reinforcement strength when calculating minimum reinforcement.
2. The behaviour of the modelled as-built grid-F wall in the Gallery Apartments Building confirmed the failure mode observed during the 22 Feb 2011 Christchurch earthquake, with a single flexural crack at the wall base and fracture of the vertical reinforcement.
3. The vertical reinforcement in the wall designed in accordance with ACI and NZS was insufficient to generate a large number of distributed cracks, resulting in premature bar fracture, and low drift capacities.
4. The performance of the wall designed in accordance to Eurocode, CSA and GB was significantly better than the walls designed according to ACI and NZS. Concentrated reinforcement in the boundary elements may improve the ductility of lightly reinforcement walls.
5. The drift capacity of concrete walls with code specified minimum vertical reinforcement improves as the aspect ratio and axial load ratio are increased.

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REFERENCES

ACI 318-11 (2011). Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary. Farmington Hills, Michigan, American Concrete Institute.

ACI 363R-84 (1984). State of the art report on high-strength concrete. Farmington Hills, MI, American Concrete Institute.

Canterbury Earthquakes Royal Commission (2012). Final report: Volume 2: The performance of Christchurch CBD buildings. http://canterbury.royalcommission.govt.nz/Commission-Reports, Wellington, New Zealand.

CEN (2004). Eurocode 8: Design of structures for earthquake resistance. Brussels, Belgium, European Committee for Standardization.

CSA (2004). Design of concrete structures. Standard CSA-A23.3-04. Toronto Canadian Standards Association.

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

GB 50010-2010 (2010). Code for design of concrete structures Beijing, China Architecture&Building Press.

Ghorbani-Renani, I., N. Velev, R. Tremblay, D. Palermo, B. Massicotte and P. Léger (2009). Modeling and testing influence of scaling effects on inelastic response of shear walls. *ACI Structural Journal* **106**(3): 358-367.

Greifenhagen, C. and P. Lestuzzi (2005). Static cyclic tests on lightly reinforced concrete shear walls. *Engineering Structures* **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* **46**(2).

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

Luu, H., I. Ghorbanirenani, P. Léger and R. Tremblay (2013). Numerical modeling of slender reinforced concrete shear wall shaking table tests under high-frequency ground motions. *Journal of Earthquake Engineering* **17**(4): 517-542.

NZS 3101:2006 (2006). Concrete Structures Standard. Wellington, New Zealand, Standards New Zealand**:** 1.

Park, R., M. J. N. Priestley and W. D. Gill (1982). DUCTILITY OF SQUARE-CONFINED CONCRETE COLUMNS. *ASCE J Struct Div* **108**(ST4): 929-950.

Sritharan, S., K. Beyer, R. S. Henry, Y. H. Chai, M. J. Kowalsky and D. K. Bull (2014). Understanding poor seismic performance of concrete walls and design implications. *Earthquake Spectra*(Available online).

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* **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* **86**(5): 582-591.

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