Experimental testing and modelling of reinforced concrete walls with minimum vertical reinforcement

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ABSTRACT: Recent research suggests that the current minimum vertical reinforcement limits in NZS 3101:2006 may be insufficient to ensure well distributed cracking in ductile plastic hinge regions. A comprehensive experimental program has been conducted to investigate the seismic behaviour of RC walls with minimum vertical reinforcement in accordance with NZS 3101:2006. The observed lateral load response, extent of crack distribution, hysteretic behaviour, failure mode, and drift capacity of one of the test wall are discussed. The experimental results confirmed that current minimum vertical reinforcing limits in NZS 3101:2006 are insufficient to form a large number of secondary cracks with the response dominated by one or two large crack in the wall base. Finite element models of tested walls has been developed and validated against the experimental data. The model was able to accurately capture both the overall response and local behaviour of the test wall presented.

1 INTRODUCTION

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 distributed cracking (Kam et al. 2011; Structural Engineering Society of New Zealand (SESOC) 2011; Bull 2012). This type of wall behaviour was also observed in the EI Faro building during the 1985 Chilean Earthquake (Wood 1989; Wood et al. 1991). The unexpected failure modes of the lightly RC walls during Canterbury Earthquake was highlighted by Canterbury Earthquake Royal Commission (CERC) (2012) which recommended that changes should be made to the New Zealand concrete structures standard, NZS 3101 (2006), to ensure yielding of reinforcement could beyond the immediate vicinity of a single primary crack.

Henry (2013) used moment-curvature analysis to investigate the minimum reinforcement requirements of NZ 3101:2006. The analysis results indicated that RC walls with minimum reinforcement in accordance with NZS 3101:2006 have only a small margin between the probable cracking moment and the nominal moment capacity of the wall section unless a significant axial load was applied. In addition, although there an extensive amount of test on reinforced concrete walls has been conducted over the last three decades, there is a lack of experimental or analytical research into flexure dominant lightly reinforced walls. To address this lack of evidence, an experimental program was conducted to evaluating the cyclic lateral-load behaviour of RC walls with minimum vertical reinforcement in accordance with NZ 3101:2006. The preliminary results from one of the tested walls are discussed along with a numerical model that was developed and validated against the test wall described.

2 EXPERIMENT INTRODUCTION

1.1 Specimen design

A summary of the main parameters for the six test walls included in the experimental program are shown in Table 1, and cross sections of the wall specimens are shown in Figure 1. 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 multi-story flexure-dominant RC walls with limited ductility in accordance with

NZS 3101:2006 (2006). The vertical reinforcement was identical for all six walls and designed using the minimum requirements in NZS 3101:2006, as shown by Eq. (1). For the test walls, the specified concrete strength ($f_c^{'}$) was 40 MPa which is the most common concrete strength used in New Zealand for multi-story buildings and the reinforcement yield strength (f_y) was 300 MPa, so the total vertical reinforcement content (ρ_n) was calculated as 0.53% using Eq. (1).

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

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

Wall	Shear span ratio *(M/Vl*$_w$*)*	Axial load ratio	Vertical reinforcement ratio (%)	Horizontal reinforcement ratio (%)	End ties (mm)
C1	2	3.5%	0.53	0.25	No
C2	4	3.5%	0.53	0.25	No
C3	6	3.5%	0.53	0.25	No
C4	2	0	0.53	0.25	No
C5	2	7%	0.53	0.25	D6@90
C6	4	3.5%	0.53	0.25	D6@60

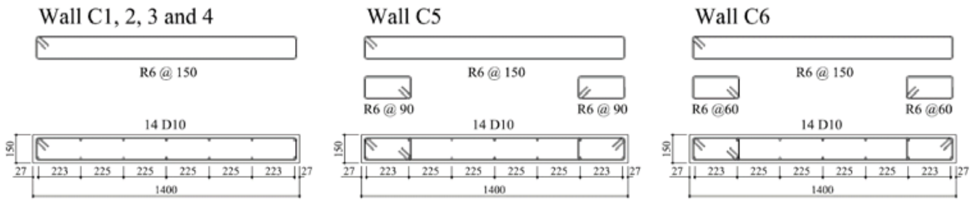


Figure 1. Cross section of test wall specimens.

The height represents the lower part 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. Three shear span ratios will be applied to the test walls, 2, 4, and 6, representing walls in a range of different building heights. The applied axial load was also varied from 0-7% of the wall axial capacity. The axial load for wall C5 triggered the NZS 3101:2006 requirement for additional confinement reinforcement in the end regions to achieve a limited ductile response. Wall C6 was identical with Wall 2 expect that stirrups to provide anti-buckling restraint were added in the wall end region.

1.2 **Test setup**

Because of the height limitation of the structural test hall in University of Auckland, 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. The test setup developed for the RC wall specimen is shown in Figure 2. One jack 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.

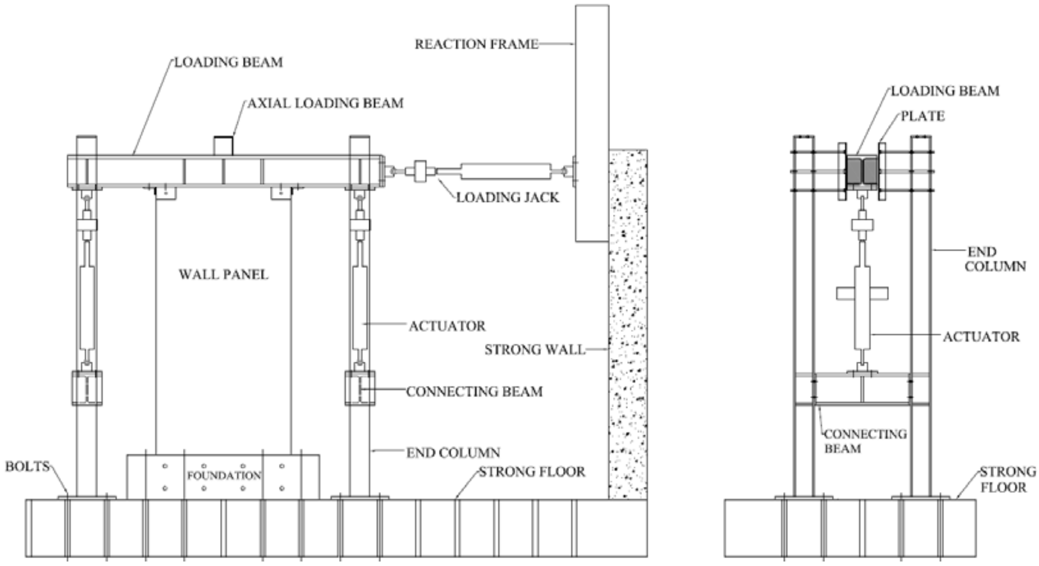
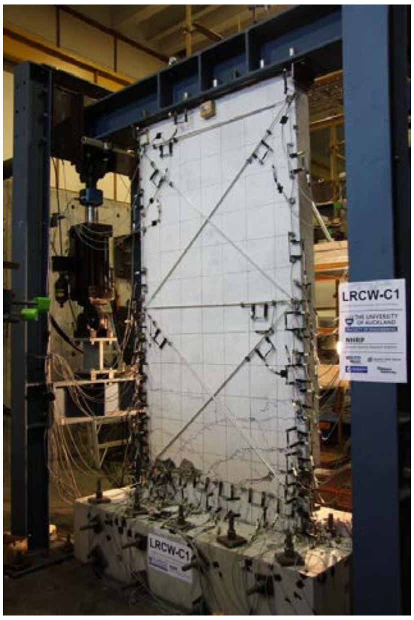


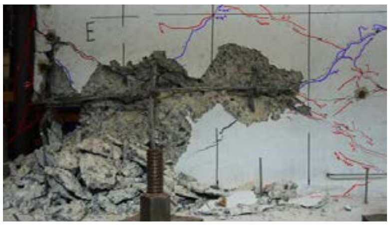
Figure 2. Experimental test setup.

3 TEST RESULT OF WALL C1

Wall C1 was the baseline wall that had the shear span ratio of 2 and axial load ratio of 3.5%. The overall condition of wall C1 at 2.5% lateral drift is shown in Figure 3-a. The wall response was dominated by flexural behaviour with 3-4 main flexural cracks forming in the lower 1/4 the wall height, as shown in Figure 3-b. These 3-4 primary flexural cracks all initiated prior to a lateral drift of ±0.25%, after which no significant new flexural cracks occurred during the test. During larger lateral drift cycles, the wall deformation was primarily concentrated at one crack at the wall base which opened up to 20 mm wide 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 at the location of that large flexural crack during cycles to lateral drift of ±1.5%. Due to the lack of confinement reinforcement, buckling of the vertical reinforcement accelerated concrete spalling and core crushing occurred during the first cycle to -2.0% drift. Two corner bars fractured during the third cycle to +2.5% drift. The failure mode of the wall in east side is shown in Figure 3-c.



(b) Extent of flexural cracking



(a) Overall condition at 2.5% drift (c) Concrete crushing and bar buckling at east end

Figure 3. Photos of wall C1 at 2.5% lateral drift.

Figure 4 shows the measured bending moment-displacement response for wall C1. The uncracked wall had a high initial cross section stiffness and the first flexural crack did not initiate until a lateral force of approximately 112.3 kN was reached, corresponding to a moment at the wall base of 332.5 kN-m, or roughly 68% of the peak strength. The inelastic response was stable up until 1.5% lateral drift when bar buckling occurred causing strength degradation on subsequent cycle. A drop of 20% of the peak 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.

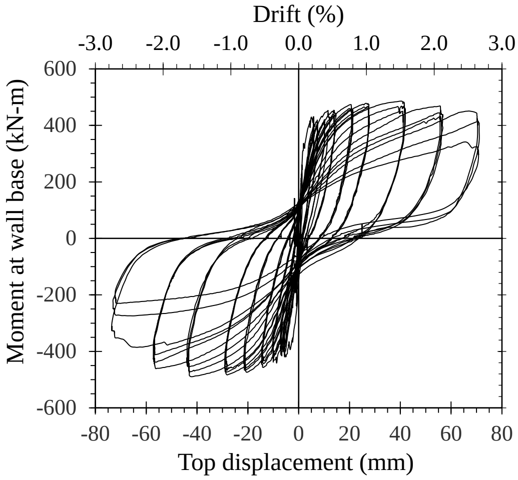


Figure 4. Lateral force-displacement response for wall C1.

4 NUMERICAL MODELLING

A numerical model was developed for the test walls using nonlinear finite element program VecTor2 (Wong et al. 2002). Four–node plane stress rectangular elements were used to model the RC walls with smeared horizontal reinforcement and two-node truss elements were used to discretely model the vertical reinforcement. The axial compression due to self-weight and gravity load actions was held constant during the analyses, whereas the lateral load applied at the top of the wall was cyclically increased in a displacement-controlled mode according to the test loading protocol. 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 trilinear stress-strain response was used for the steel reinforcement. The hysteric response of the concrete material incorporated plastic offset and unloading paths using the Palermo and Vecchio model (Palermo and Vecchio 2002). Reinforcement bond-slip was not considered as the failure of the test wall was not governed by bond slip failure and it was found that inclusion of a bond slip model has not significant influence of the model results. Detailed descriptions of the material models can be found in the VecTor2 user manual (Wong et al. 2002).

A comparison of the measured and calculated base moment- displacement responses for wall C1 is shown in Figure 6. The finite element model captured the measured response of the wall reasonably well. The lateral load capacity and the lateral stiffness of the wall are well represented for most of the lateral drifts. During the test, degradation in the measured lateral capacity and stiffness of the wall occurred during cycles to 2.0% and 2.5% drift as the vertical reinforcement buckled. The VecTor2 model did consider bar buckling using a model based on compression stress in the reinforcement, but this was insufficient to capture the buckling observed during the test. The test results highlighted that the vertical reinforcement in the test walls was particularly vulnerable to buckling because the wide cracks that form in these walls increases the concentration of tensile strains in the vertical reinforcement and it easily buckled on the reverse cycle.

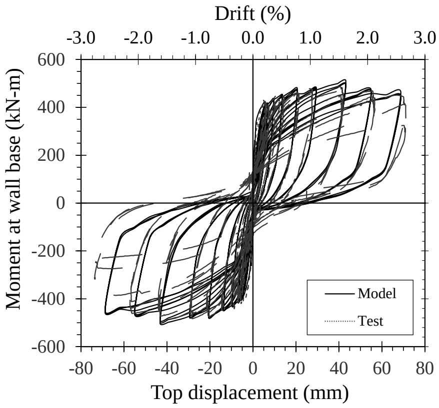
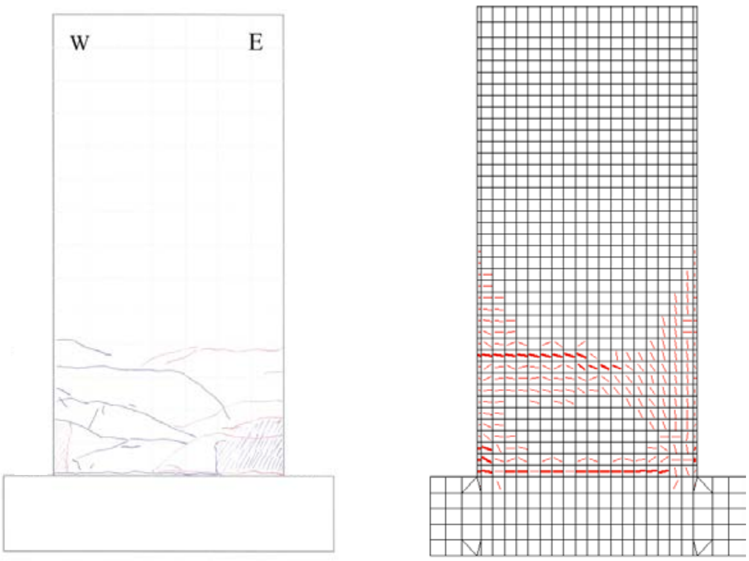


Figure 5. Comparison of hysteric responses between test and model.

The crack pattern documented during the C1 wall test is compared against the crack pattern of the modelled wall at 1.5% lateral drift in Figure 6. During the test, the behaviour of wall C1 was controlled by 3-4 main primary cracks with the bottom crack opening the widest and dominative the response. In the model, the crack pattern was also controlled by a small number of primary cracks with the bottom primary crack opening significantly wider than the second primary crack. During both in the test and the model, the top lateral deformation was mostly attributed to the wide crack at the wall base. These results indicated that the model can predict the behaviour of lightly walls with only a limited number of flexural cracks.

Comparison between the measured and modelled steel strain distribution over the wall height at 1.5% lateral drift is shown in Figure 7. The strain measured during the test was based on the gauge length of 150 mm. Therefore, the strain extracted from the model was also the average strain over a 150 mm mesh length. When considering that the experimental results were often affected by various experimental and specimen uncertainties, such as the crack distribution, the model results are considered to be in reasonable agreement with the measured strains. The test result showed that instead of the steel tensile strains yielding consistently over the entire plastic hinge region, large spikes in the reinforcement strains were observed at location of wide flexural cracks. As shown in Figure 7, the model was also able to capture this irregular strain profile. The peak strain was slightly overestimated by the model, but these differences are considered acceptable knowing that a small difference in bar stress produces a large variation in strain in the post-yield region.



(a) Finial crack pattern of test wall (b) Finial crack pattern of model wall

Figure 6. Comparison of crack patterns between test and model.

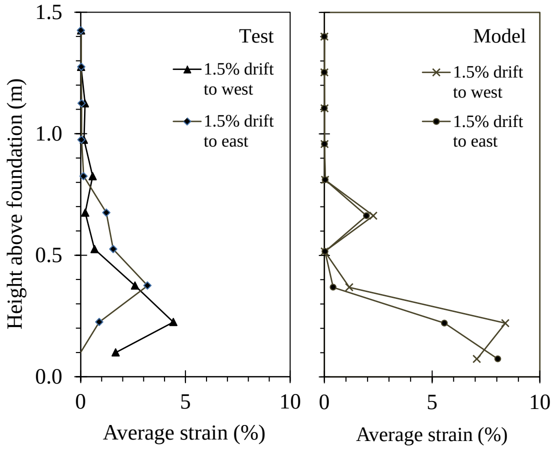


Figure 7. Comparison of steel strain distribution between test and model.

5 CONCLUSION

An experimental program designed to evaluating minimum vertical reinforcement limits in NZ 3101:2006 was described and the test result of one of the test walls was discussed. A numerical model of the test walls was developed and validated against test data. Based on the analysis the following conclusions were drawn:

1. The experimental results confirmed that current minimum vertical reinforcement limits in NZS 3101:2006 are insufficient to ensure that a large number of secondary cracks develop. The lateral deformation of the test wall was mostly contributed at 1-2 wide cracks at the wall base.
2. The finite element model captured both the overall response and local behaviour of the wall with good accuracy when considering the cyclic hysteresis response, crack pattern and vertical reinforcement strains. Buckling of the vertical enforcement was not well captured by the model because the large tensile strains are not accounted for.

6 ACKNOWLEDGMENTS

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