Testing of RC walls to investigate proposed minimum vertical reinforcement limits in NZS 3101:2006 (A3)

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ABSTRACT: During the 2010/2011 Canterbury Earthquakes, several reinforced concrete (RC) walls in multi-storey building formed a limited number of cracks in the plastic hinge region as opposed to expected distributed cracking. Previous tests and numerical models have confirmed that RC walls with distributed minimum vertical reinforcement in accordance with existing provisions in NZS 3101:2006 (A2) are unlikely to form a large number of secondary cracks, and are only suitable for walls designed for low ductility demands. As a result, new amendment have been proposed to the minimum vertical reinforcement limits in NZS 3101:2006 that require additional vertical reinforcement to ensure well distributed secondary cracking in ductile walls. A series of large-scale RC walls have been tested to investigate the seismic performance of RC walls with additional reinforcement at the end regions of the wall in accordance with these proposed amendments in NZS 3101:2006 (A3 - draft). The test walls were all designed to represent flexural dominant RC walls in multi-storey buildings with the bar diameter and number of reinforcement bars in the end region of the wall varied. The observed lateral load response, extent of crack distribution, hysteretic behaviour, failure mode, and drift capacity of two of the tested walls are discussed. As expected the increased vertical reinforcement in the ends of the test walls resulted in a significant increase in the number and distribution of cracks in the plastic hinge region when compared to the previously tested walls that had only minimum distributed reinforcement.

1 INTRODUCTION

During the 2010/2011 Canterbury earthquakes in New Zealand, several lightly RC walls in multi-storey buildings designed for ductility formed only 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; Sritharan et al. 2014). In response to the observed performance of lightly RC walls, researchers questioned whether the current minimum vertical reinforcement requirements for ductile RC walls are sufficient to generate a large number of cracks in the plastic hinge region (Henry 2013). Lu et al. (2015) conducted a series of tests on six RC walls designed in accordance with the current minimum vertical reinforcement requirements in New Zealand Concrete Structures Standard NZS 3101: 2006 Amendment 2 (A2) (2006). The test results confirmed that RC walls designed with minimum allowable distributed vertical reinforcement are unlikely to form a large number of secondary cracks in the plastic hinge region, with the behaviour of the test walls controlled by 1-3 large primary flexural cracks at the wall base. More recently, Lu and Henry (2015) developed a detailed finite element model for lightly RC walls, and the results of parametric study showed that the behaviour of walls designed with minimum allowable distributed vertical reinforcement in NZS 3101: 2006 (A2) was significantly influenced by wall size, reinforcement properties and concrete strength. Based on the findings of the modelling, it was concluded that the drift capacity of the scaled walls tested by Lu et al. (2015) may overestimate the drift capacity of a full scale as-built RC wall.

The current minimum vertical reinforcement requirements for RC walls in NZS 3101:2006 (A2) are sufficient to prevent a sudden loss in strength after first cracking, however, they are insufficient to ensure that a large number of secondary cracks form in the plastic hinge region and only applicable for nominally ductile walls. As a result, new amendments have been proposed to the minimum vertical reinforcement limits in NZS 3101:2006 Amendment 3 (A3-draft) (2015). The proposed amendment requires the ductile walls require additional minimum vertical reinforcement at the ends of the wall to ensure well distributed secondary cracking.

A series of large-scale RC walls are currently being tested to investigate the seismic performance of RC walls with additional reinforcement at the wall end regions in accordance with the proposed amendments in NZS 3101:2006 (A3-draft). The preliminary experimental results of two tested walls including crack pattern, failure mode and overall hysteric response are presented.

2 EXPERIMENT INTRODUCTION

2.1 Specimen design

A summary of the main parameters for the test walls is shown in Table 1, and drawings of the wall specimen are shown in Figure 1. All the test walls had identical dimensions to the six test walls tested previously by Lu et al. (2015), consisting of a length of 1.4 m, a height of 2.8 m and a thickness of 150 mm. The shear span ratio was 4 and the axial load was 3.5% corresponding to actual axial loads applied during the test of 294 kN, which were kept consistent as that of wall C2 in the phase I test by Lu et al. (2015). The test walls were designed to approximately represent a 40-50% scale version of multi-storey flexure-dominant RC walls with full ductile detailing requirements in accordance with NZS 3101 (2006). The full scale prototype wall was expected to have a thickness of 300-375 mm, a length of 2.8-3.5 m and a height of 12-24 m (4-8 stories), which was comparable to the dimension of the grid-3 wall in the Gallery Apartments Building in Christchurch (Smith and England 2012).

The vertical reinforcement was designed using G300E reinforcement and a specified concrete strength ($f_c^{'}$) was assumed to be 30 MPa, resulting in a minimum required vertical reinforcement ratio in the end region and central web region of the wall of 0.91% and 0.46% using Eq. (1) and Eq. (2), respectively. The length of the end region for all the test walls was calculated as 210 mm using $0.15l_w$ in accordance with NZS 3101:2006 (A3-draft).

$$\rho_{le} \geq \frac{\sqrt{f_c^{'}}}{2 f_y} \tag{1}$$

$$\rho_{l} \geq \frac{\sqrt{f_c^{'}}}{4 f_y} \tag{2}$$

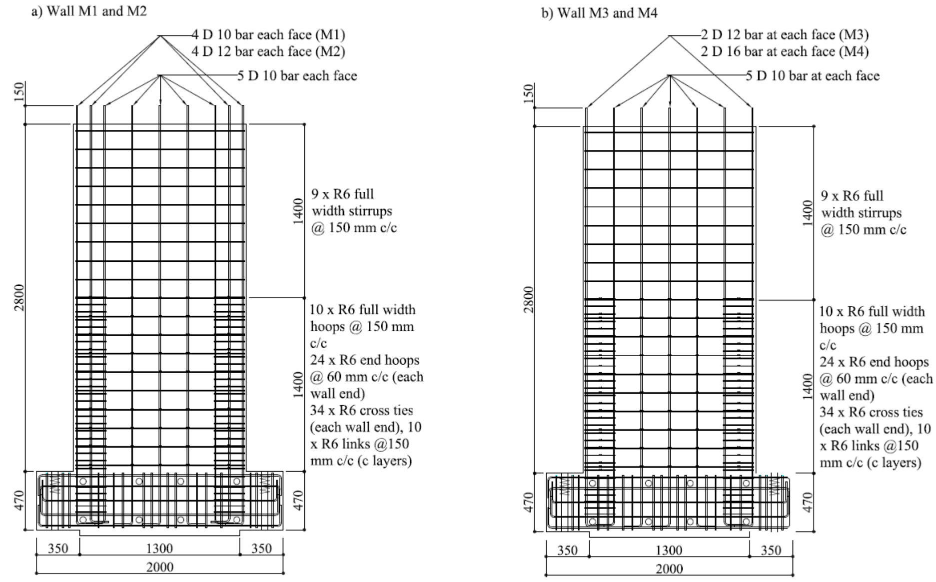
All the four walls were identical expect the end zone vertical reinforcement ratio was varied from 0.72% to 1.44%. The distributed reinforcement ratio for all the four walls was 0.465%, resulting two layers of five D10 bars placed at 225 mm centers over the central wall web region. Wall M1 was designed to closely satisfy the proposed new minimum vertical reinforcement requirements which had a lumped reinforcement ratio of 1.0%, resulting in four D10 placed at wall end region. The end zone vertical reinforcement ratio of wall M2 was 1.44% consisting of four D12 bars which was higher than the proposed end region minimum vertical reinforcement. Wall M3 did not satisfy the proposed minimum vertical reinforcement requirements in the end zone, and was designed to investigate either a reduced end zone reinforcement ratio (0.72%) or a smaller end zone length (150 mm), as shown in Figure 1. Wall M4 had a similar end zone vertical reinforcement ratio as wall M1, but used two D16 reinforcement bars instead of four D10 bars to investigate using larger diameter bars right at the wall end.

The horizontal reinforcement was designed following procedures in NZS 3101 (A3-draft) (2015). Only minimum horizontal reinforcement was required to satisfy shear capacity, resulting in D6 stirrups distributed evenly at 150 mm centers over the wall height, as shown in Figure 1. According to NZS 3101 (A3-draft) (2015), anti-buckling ties are required within the compression neutral axis depth for ductile walls and the spacing of the ties should be less than $10d_b$. To meet this ductility requirement, D6 stirrups were placed in the wall toes at 60 mm centers over the lower 1.4 m of the wall height for all the test walls, as shown in Figure 1. Cross ties were also placed between the vertical reinforcement in the wall web region at 150 mm centers over the lower 1.4 m of the wall height.

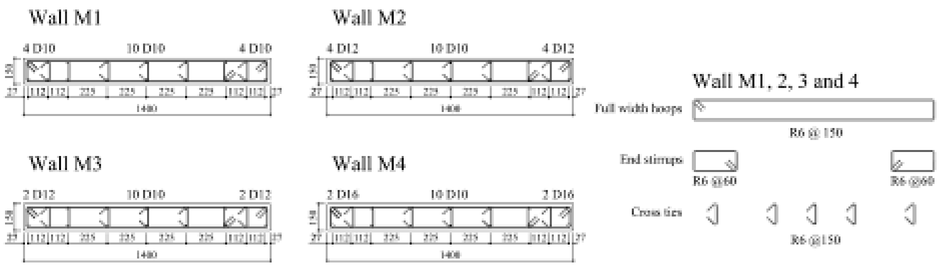
To simulate the effect of long-term average concrete strengths, a 40 MPa concrete mix was used for all of the walls despite the specified concrete strength being 30 MPa.

Table 1. Details of test walls

Wall	Shear span ratio	Axial load ratio	Vertical reinforcement ratio (%)			End region length (mm)	Horizontal reinforcement ratio (%)	End ties (mm)	Web ties (mm)
			End region	Web region	Total				
M1	4	3.5%	1.00	0.465	0.673	210	0.25	R6@60	R6@150
M2	4	3.5%	1.44	0.465	0.804	210	0.25	R6@60	R6@150
M3	4	3.5%	0.72 (1.0)	0.465	0.589	210 (150)	0.25	R6@60	R6@150
M4	4	3.5%	1.28	0.465	0.757	210	0.25	R6@60	R6@150



(a) Elevations



(b) Cross sections

Figure 1 - Drawings of the test walls

2.2 **Test setup**

The test setup used is shown in Figure 2. One actuator was 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 combination of moment and axial load at the top of the wall. The bending moment distribution over the wall height was presented by Lu et al. (Lu et al. 2014).

Figure 2 - Test setup and moment distribution over the wall height

3 TEST RESULT OF WALL M1 AND M2

Table 2 provides a summary of the drift cycle during which key observations were made during the tests of walls M1 and M2, including first cracking, concrete spalling, reinforcement buckling, core concrete crushing and reinforcement fracture. The overall conditions of the two walls at the end of each test are shown in Figure 3. The crack patterns as well as the maximum measured crack widths at the end of two current tested walls and one previous test wall C2 are shown in Figure 4. It should be noted that these crack pattern figures were drawn from the southern side, so the left side is west and right side is east. The final condition and the moment-displacement hysteresis response for the two tested walls are shown in Figure 5 and Figure 6, respectively. In the following section, a brief description of the evolution of the test wall behaviour is given for walls M1 and M2.

3.1 Wall M1

Wall M1 was considered the baseline wall with an end region vertical reinforcement ratio of 1.0%, which closely satisfies the minimum requirement proposed in NZS 3101:2006 (A3-draft). It should be noted that wall M1 was cracked at left side prior to test due to the error of the left vertical actuator. Nevertheless, the behaviour was not influenced significantly except for the initial stiffness being lower than expected. The wall response was dominated by flexural behaviour with a large number of horizontal cracks extending over almost the entire wall height. Compared to the previously tested wall C2 which had minimum distributed reinforcement as per NZS 3101: 2006 (A2) and was dominated by 3-4 large cracks (Lu et al. 2015), the cracks in wall M1 were more evenly distributed over the plastic hinge region. As shown in Figure 4-a, wall M1 had more cracks and a smaller crack spacing compared to wall C2. The maximum crack width at wall failure was around 6 mm, which was significantly less than the large 20 mm crack width for wall C2. The wall deformation was attributed to all the cracks over the plastic hinge length rather than one single crack at wall base. Furthermore, unlike wall C2 in which all the flexural cracks formed prior to 0.5% lateral drift, new secondary cracks formed in wall M1 during cycles up to ±1.5% lateral drift.

The concrete at the corners of the wall started to spall and buckling of the vertical reinforcement initiated during cycles to lateral drifts of ±2.0%. The reinforcement buckling occurred during later drift cycles than wall C6 tested in Phase I which also had R6 stirrups placed at 60 mm centers in the wall end region. Increasing the vertical reinforcement resulted in an increased number of secondary cracks allowed the reinforcement strains to be more evenly distributed over the plastic hinge region.

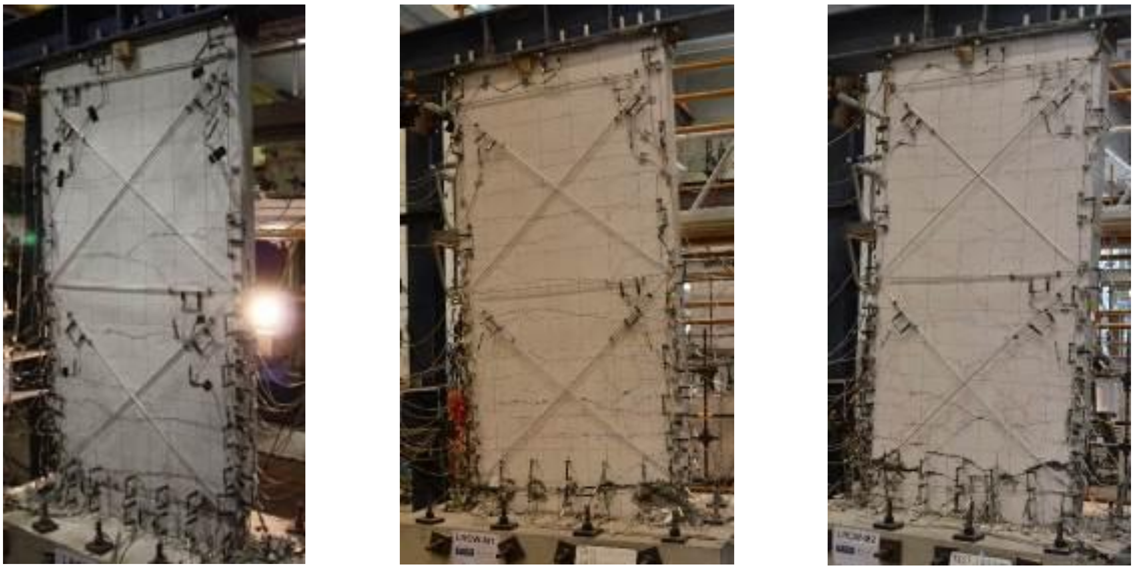
This even distribution of plasticity helped delay buckling of the vertical reinforcement as the vertical reinforcement strain were not concentrated at several wide cracks as was the case for the Phase I walls. The two buckled reinforcing bars at the west end fractured during the second and third cycle to +2.5% lateral drift, respectively. At east end, one reinforcing bar fractured during the third cycle to -2.5% lateral drift. The final condition on the west and east end of the wall was significantly better than that of wall C2 at same drift of 2.5%, as shown in Figure 5-a.

The initial cross section stiffness of wall M1 was lower than expected due to the unexpected cracking before the test, as shown in Figure 6-a. The inelastic response of wall M1 was stable up until 2.0% lateral drift when buckling of the vertical reinforcement caused strength degradation on subsequent cycle. A drop of 20% of the peak strength occurred when the buckled reinforcing bar fractured during the third cycle to +2.5% lateral drift.

Table 2. Key observations of all six test walls

Test wall	Direction	First cracking	Concrete spalling	Bar buckling	Core concrete crush-ing	Bar fracture
M1	+	N/A	+2.0%[1 a]	+2.0%[3]	+3.5%[3]	+2.5%[2]
	-	N/A	-2.0%[1]	-2.0%[2]	-3.5%[1]	-2.5%[3]
M2	+	+0.2%	+2.0%[3]	+2.5%[3]	+3.5%[1 b]	+3.5%[1]
	-	-0.2%	-2.0%[3]	-2.5%[2]	-3.5%[2]	-3.5%[3]

[a] superscript present the cycle number, [b] instability occurred



(a) wall C2 (b) wall M1 (c) wall M2

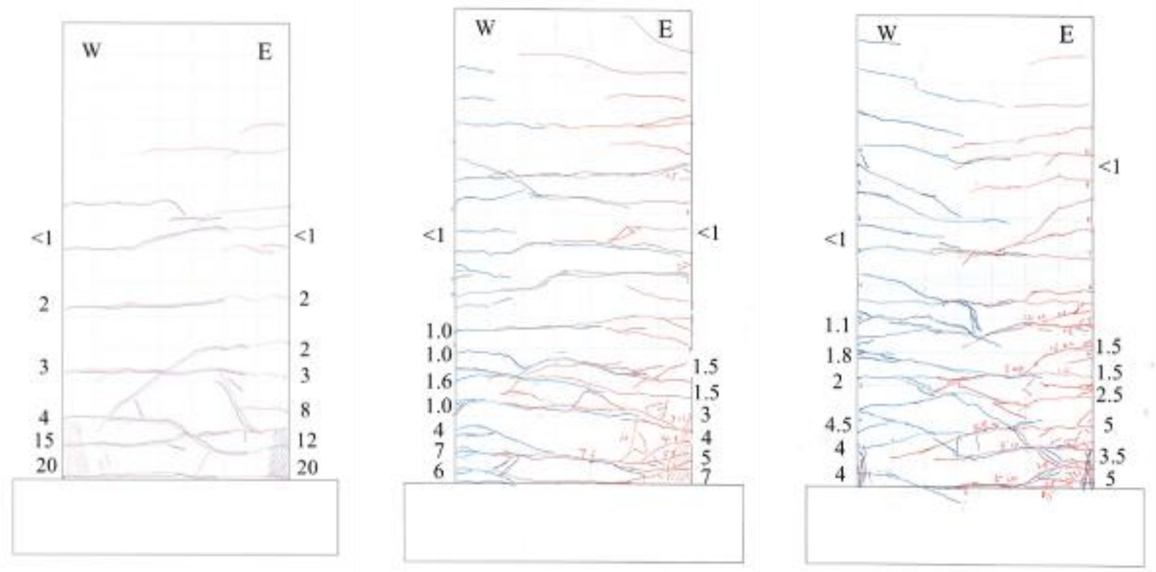
Figure 3 - Overall condition of the tested walls

3.2 Wall M2

Wall M2 was identical to wall M1 except for a larger end region reinforcement ratio of 1.44% corresponding to four D12 reinforcing bars (replacing D10 bars used for wall M1). Similar to wall M1, the behaviour of wall M2 was dominated by flexure with a large number of cracks occurring over the full wall height. As shown in Figure 4-b, more obvious inclined shear cracks were observed in the central wall web region and more secondary cracks occurred at wall end region due to the larger end region reinforcement ratio. New secondary cracks formed up until drift cycles to ±1.5%. In the later stages of the test, the inclined web cracks was wider than the cracks at wall edge due to the difference in the distributed reinforcement ratio in wall web and reinforcement ratio at wall end region.

Due to the stability of larger diameter reinforcing bars, concrete spalling and reinforcement buckling of wall M2 was delayed when compared to wall M1, and initiated during cycles to ±2.0% lateral drift and ±2.5% lateral drifts, respectively. At west end, the buckled reinforcing bar fractured during the first cycle to +3.5% lateral drift. Subsequent reinforcing bars in the end region fractured during the next two cycles to +3.5% lateral drift. At east end, the wall instability occurred in the end region during the first cycle to +3.5% lateral drift. Reinforcing bar fracture did not occur at the east end until the third cycle to -3.5% drift. The final condition on the west and east end of wall M2 is shown in Figure 5-b.

The first flexural crack initiated during the first cycle to +0.2% lateral drift with a wall base moment of 294.6 kN-m, or roughly 43% of the peak strength. As shown in Figure 6-b, the inelastic response of wall C2 was stable up until ±2.5% lateral drift when buckling of the vertical reinforcement occurred and caused a gradual degradation in wall strength. Three vertical reinforcing bars fractured during the second cycle to +3.5% lateral drift, leading a 20% drop of peak strength.



(a) wall C2 (B) wall M1 (C) wall M2

Figure 4 - Crack pattern of the tested walls


west end west end


east end east end

(a) wall M1 (b) wall M2

Figure 5 - Failure modes of the two tested walls

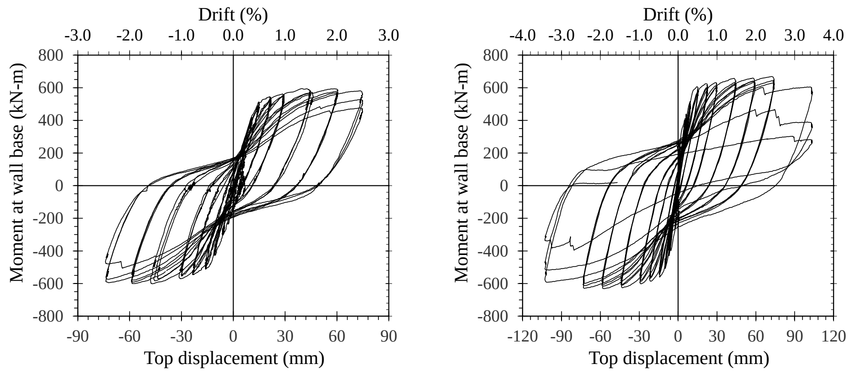


Figure 6 - Moment-displacement response of the two tested walls

4 CONCLUSIONS

An experimental program designed to investigate the proposed amendments to minimum vertical reinforcement limits in NZS 3101:2006 (A3-draft) was described and the test results of two of the tested walls were presented. Based on these initial test results the following conclusions were drawn:

1. The increased vertical reinforcement in the wall end region in accordance with NZS 3101:2006 (A3-draft) resulted in a significant increase in the number and distribution of cracks in the plastic hinge region when compared to the previously tested walls that had minimum distributed reinforcement.
2. Increasing the vertical reinforcement content in the end region also helped to delay reinforcement buckling due to the increased number of secondary cracks allowed the reinforcement strains to be more evenly distributed over the plastic hinge region.

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