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EVALUATION OF MINIMUM VERTICAL REINFORCEMENT LIMITS FOR REINFORCED CONCRETE WALLS

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SUMMARY

During the 2010/2011 Canterbury earthquakes, several lightly reinforced concrete walls in multi-storey buildings formed only a limited number of cracks in the plastic hinge region as opposed to the expected distributed cracking. In response to this, the Canterbury Earthquakes Royal Commission (CERC) highlighted the need for further research to improve the seismic design of lightly RC walls. A total of six test walls were tested to investigate the seismic behaviour of RC walls with distributed minimum vertical reinforcement in accordance with current provisions in NZS 3101:2006. The experimental results confirmed that current minimum vertical reinforcing limits in NZS 3101:2006 are insufficient to form a large number of secondary cracks and are only suitable for walls designed for low ductility demands. Detailed numerical models of lightly RC walls were also developed to understand the behaviour of the test walls, and to conduct additional analyses to investigate the performance of walls with minimum vertical reinforcement. Results from these additional analyses showed that wall size, reinforcement type and concrete strength had a significant effect on the cracking behaviour and lateral drift capacity of walls that satisfied the current minimum reinforcement limits in NZS 3101:2006. A second phase of the tests are currently underway to investigate the seismic performance of RC walls with additional reinforcement at the end regions of the wall, in accordance with the proposed amendments to minimum vertical reinforcement requirements for ductile RC walls in NZS 3101:2006 (amendment 3).

INTRODUCTION

During 2010/2011 Canterbury earthquakes, severe damage was observed to reinforced concrete (RC) walls in several modern multi-storey buildings (Sritharan et al. 2014). In particular, several lightly reinforced concrete walls in multi-storey buildings 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). This behaviour can cause numerous problems, such as premature fracture of vertical reinforcement, low drift capacity, large axial elongations, wall sliding, out-of-plane wall instability, and local wall buckling. In response to the observed performance of lightly reinforced RC walls, the Canterbury Earthquake Royal Commission (2012) recommended that research be conducted to investigate crack control for RC walls, and that changes should be made to the New Zealand Concrete Structures Standard (NZS 3101) (2006) to ensure that yielding of reinforcement can extend beyond the immediate vicinity of a single primary crack.

A research program was initiated to understand performance of lightly RC walls damaged during the Canterbury earthquakes and to improve concrete design standard (NZS 3101: 2006). A combination of experimental testing and numerical modelling has been used to investigate the seismic behaviour of RC walls designed with minimum vertical reinforcement as per the current requirements in NZS 3101: 2006. A second phase of the tests are currently underway to investigate the seismic performance of RC walls with additional reinforcement at the end regions of the wall, in accordance with the proposed amendments to minimum vertical reinforcement requirements for ductile RC walls in NZS 3101:2006 (amendment 3).

Experimental evaluation of current minimum vertical reinforcement limits

The experimental program comprised of six large-scale RC cantilever test walls that were subjected to pseudo-static cyclic loading. The geometry of the test walls, test setup, and the main conclusions from the tests are summarised below, and more detailed test results have been published previously (Lu and Henry 2015; Lu et al. 2015a; Lu et al. 2015b).

Test specimens and setup

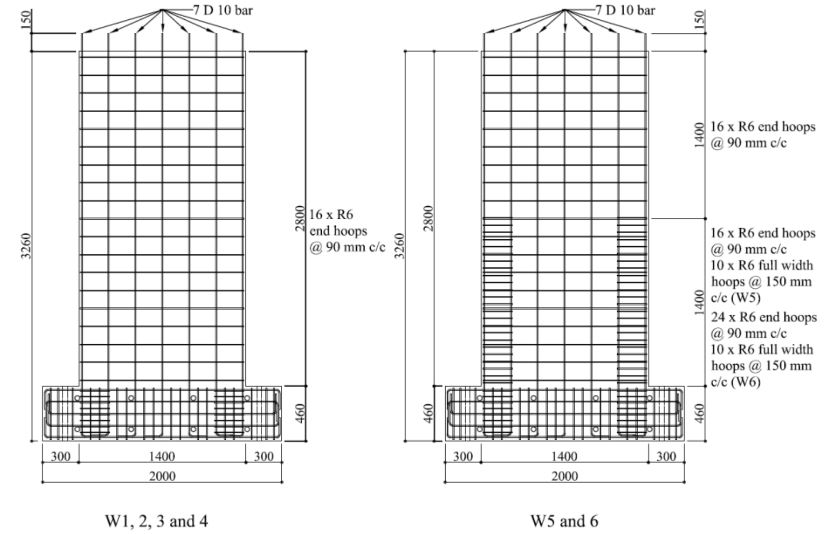
A summary of the six test walls is shown in Table 1, and drawings of the wall specimen are shown in Figure 1. The 1.4 m long, 2.8 m high and 150 mm thick wall specimen were designed to approximately represent a 40-50% scale version of RC walls with limited ductility in accordance with NZS 3101:2006. 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 and the reinforcment yield strength (f_y) was 300 MPa, so the total vertical reinforcement content (ρ_n) was calculated as 0.53% using Eq. 1. Three shear span ratios (M/VL_w) were 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 between 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.

$$\rho_n \geq \frac{\sqrt{f_c^{'}}}{4 f_y} \quad (1)$$

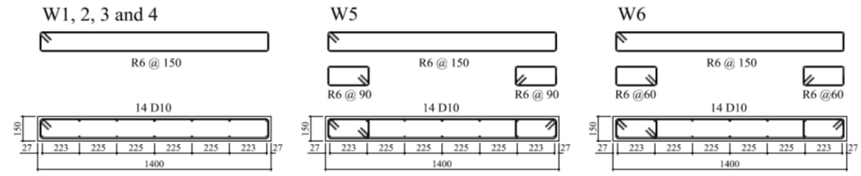
Table 1. Details of six test walls in Phase I

Wall	M/VL_w ratio	Axial load ratio	Specified material properties: $f_c^{'}$ (MPa)	f_y (MPa)	Vertical reinforcement ratio (%)	End stirrups (mm)
C1	2	3.5%	40	300	0.53	No
C2	4	3.5%	40	300	0.53	No
C3	6	3.5%	40	300	0.53	No
C4	2	0	40	300	0.53	No
C5	2	7%	40	300	0.53	D6@90
C6	4	3.5%	40	300	0.53	D6@60

The test setup developed for the RC wall specimen is shown in Figure 2. The walls were constructed as precast elements with the two foundation blocks located on each side of the wall. A 15 mm gap between the two foundation blocks and the wall panel was grouted prior to post-tensioning the components together to the laboratory strong floor to create the foundation. The vertical reinforcement at the top of the wall panel was attached to a steel loading beam. Because of height limitations in the structural test hall, a test setup was designed to simulate the expected seismic loading on the bottom two storeys of a 40-50% scaled wall from a multi-storey building. An actuator was attached between the steel loading beam and the strong wall to apply horizontal loads to the wall, and two additional actuators were attached vertically at each end of the wall to achieve the required moment and axial load at the top of the wall.



(a) Elevations



(b) Cross sections

Figure 1. Details of test walls

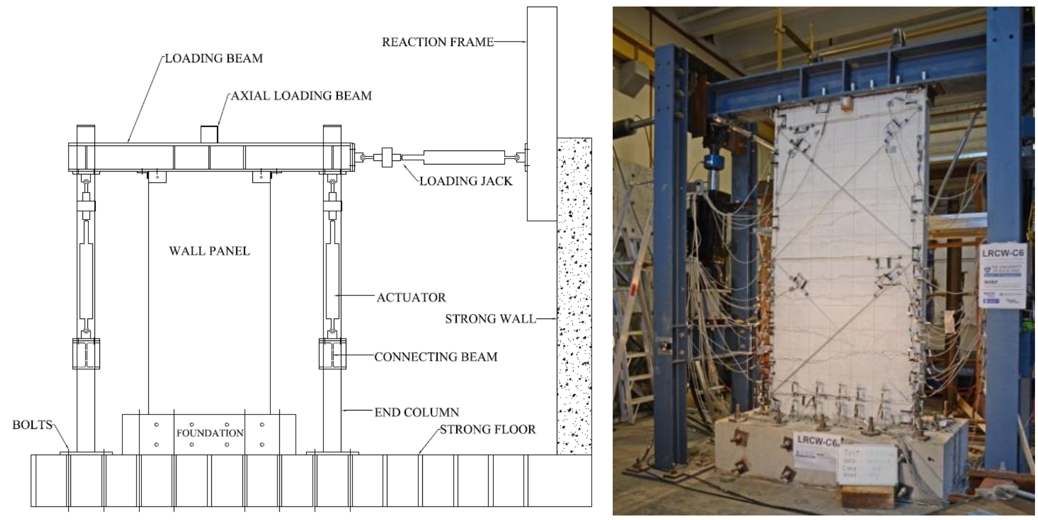
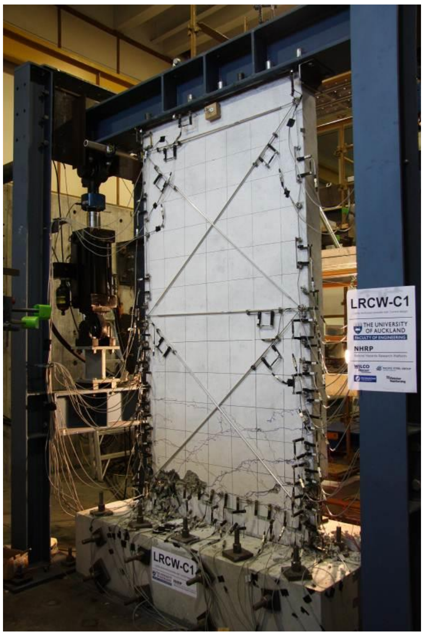


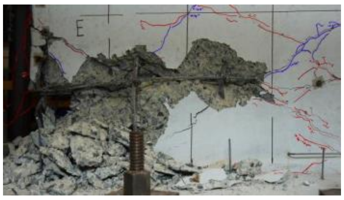
Figure 2. Test setup for RC walls

Results and conclusion from the test

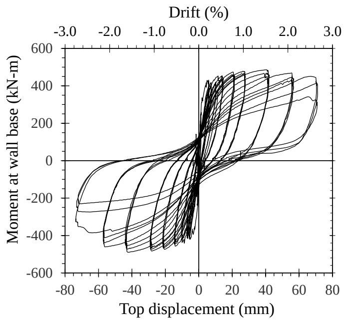
The behaviour of all six test walls was controlled by 1-3 large flexural cracks at the wall base, as shown for wall C1 in Figure 3-a. The failure for all the six test walls was controlled by vertical reinforcement buckling and subsequent reinforcement fracture, for example the failure of wall C1 is shown in Figure 3-b. The lateral drift capacity of the wall with no axial load was 1.5% and other five walls were all 2.5%, as shown an example of wall C1 in Figure 3-c.The curvature and reinforcement strain distributions in the plastic hinge region of the test walls were not smooth and with large concentrations of curvature and strain at the locations of dominant flexural cracks. This concentration of deformation at 1-3 flexural cracks greatly reduced the spread of the plasticity. Both the shear span ratio and inclusion of transverse reinforcement ties in the ends of the wall had no significant influence on the drift capacity of the test walls.

Based on the test results it was concluded that the current minimum vertical reinforcement requirements for RC walls in NZS 3101:2006 (amendment 2) are sufficient to prevent a sudden loss in strength after first cracking, however, they are insufficient to ensure that a large number of well distributed secondary cracks form in plastic hinge regions. Furthermore, lightly reinforced concrete walls are particularly vulnerable to reinforcement buckling at modest lateral drifts. To achieve improved ductility for lightly RC walls, the minimum vertical reinforcement requirements for RC walls need to be revised to include criteria that ensure well distributed primary and secondary cracks develop in plastic hinge regions. Recommendations were made to the NZS 3101 technical committee that resulted in the proposed increased vertical reinforcement in the end region of ductile walls (COOK et al. 2014).





(b) Concrete crushing and bar buckling at east end



(a) Overall condition at 2.5% drift

(c) Moment-displacment repsonse

Figure 3. Test results of wall C1

Numerical modelling evaluation of current minimum vertical reinforcement limit

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). The back-bone of the steel model included an initial linear-elastic response, a yield plateau, and a non-linear strain hardening phase until rupture. 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 had no significant influence of the model results. Detailed descriptions of the material models can be found in the VecTor2 user manual (Wong et al. 2002). The proposed model was able to capture 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 (Lu and Henry

2015).

Ten wall designs were modelled to investigate the effect of wall dimension, reinforcement type and concrete strength on the lateral load and cracking behaviour. Details of the modelled walls are shown in Table 2 and the calculated moment-displacement responses for each model are shown in Figure 4. The results of walls C1-1 and C1-2 highlighted that there is a significant size effect for lightly reinforced concrete walls with the lateral drift capacity decreasing greatly as the wall dimensions increased. This size effect was expected as the wall failure was controlled by bar fracture and as the wall length increased the hinge rotation at the maximum crack width decreased. Using reinforcement with higher strength and lower ductility (walls C1-3, C1-4, C1-5) did not significantly impact the crack pattern, but did greatly decrease the lateral drift capacity of the walls as reinforcement fractured at smaller crack widths. Furthermore, reducing the strain hardening ratio of the reinforcement (walls C1-6, C1-7) and increasing the concrete strength (walls C1-8, C1-9, C1-10) both resulted in a reduction in secondary cracking over the plastic hinge region and a reduced lateral drift capacity due to the decreased ratio between reinforcement tension force and concrete tensile strength.

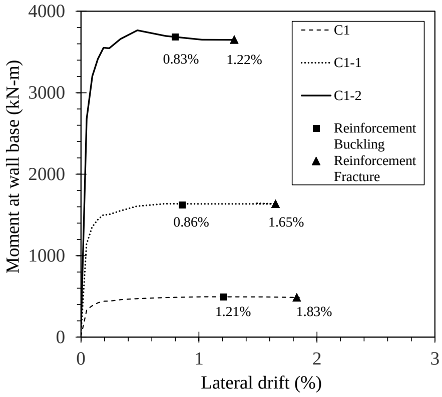
Table 2 Details of modelled walls

Wall No.	Main parameter	Dimension (mm)	Reinforcement properties						Concrete	
			Type	f_y (MPa)	f_u (MPa)	f_u/f_y	ε_{cu} (%)	Yield plateau	f'_c (MPa)	f_t (MPa)
C1		150 x 1400 x 2800	G300E	300.0	409.0	1.36	18.1	No	38.5	2.88
C1-1	Dimension	225 x 2100 x 4200	G300E	300.0	409.0	1.36	18.1	No	38.5	2.88
C1-2		300 x 2800 x 5600	G300E	300.0	409.0	1.36	18.1	No	38.5	2.88
C1-3	Reinforcement type	150 x 1400 x 2800	G500E	300.0	409.0	1.20	12.4	No	38.5	2.88
C1-4		150 x 1400 x 2800	G500E	544.4	653.3	1.20	12.4	Yes	38.5	2.88
C1-5		150 x 1400 x 2800	Class C	601.0	725.5	1.21	7.7	No	38.5	2.88
C1-6	Strain hardening ratio	150 x 1400 x 2800	G300E	300.0	345.0	1.15	18.1	No	38.5	2.88
C1-7		150 x 1400 x 2800	G300E	300.0	450.0	1.50	18.1	No	38.5	2.88
C1-8	Concrete strength	150 x 1400 x 2800	G300E	300.0	409.0	1.36	18.1	No	40.0	3.51
C1-9		150 x 1400 x 2800	G300E	300.0	409.0	1.36	18.1	No	50.0	4.07
C1-10		150 x 1400 x 2800	G300E	300.0	409.0	1.36	18.1	No	60.0	4.60

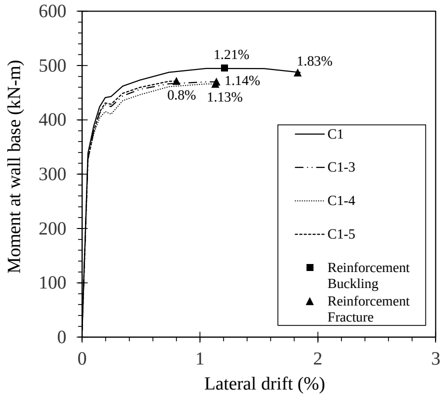
Proposed experimental evaluation of new minimum limits

Based on the observations from the Canterbury earthquakes and the initial research findings presented, amendments were proposed to the minimum vertical reinforcement limits in NZS 3101:2006. The requirements are as follows as described by Russell et al. (2015)

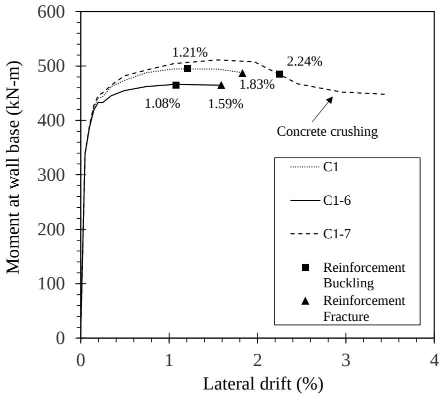
- The distributed vertical reinforcement ratio should be larger than $\sqrt{f'_c}/4f_y$ in all walls.
- The vertical reinforcement ratio in the end region of walls with limited ductile or ductile plastic hinge regions should be larger than $\sqrt{f'_c}/2f_y$. This was to ensure that secondary cracks would form even considering expected concrete tensile strengths.
- The length of the boundary element was proposed to be 0.15lw for rectangular walls.
- The web (central) vertical reinforcement ratio should be at least 30% of the reinforcement ratio at the ends of the wall to ensure that the distributed secondary cracks propagate through the web region.



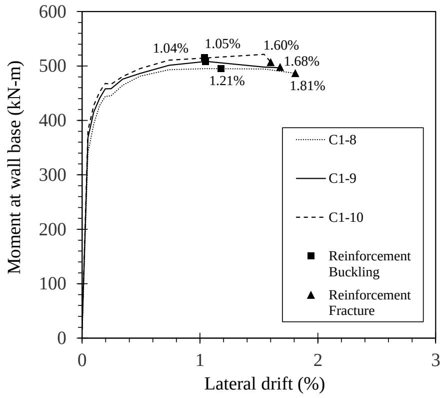
(a) size effect



(b) reinforcement type



(c) strain hardening ratio



(d) concrete strength

Figure 4 Comparison of moment-drift curves for modelled RC walls

Four test walls were designed to verify and refine these proposed requirements. The details of the test walls are listed in Table 3, and the drawings are shown in Figure 5. All the test walls had identical dimensions to the six phase I test walls described earlier. The shear span ratio for all walls was 4 and axial load ratio was 3.5%. The vertical reinforcement was designed considering expected concrete tensile strength using G300E reinforcement and 40 MPa concrete. However, the specified concrete strength (f_c') was assumed to be 30 MPa so the minimum required vertical reinforcement ratio in the end zone and central web region of the wall were calculated as 0.91% and 0.46% respectively. All the four walls were identical except the end zone vertical reinforcement ratio was varied from 0.72% to 1.44%. Wall M1 was designed to closely satisfy the proposed minimum vertical reinforcement requirements, and the end zone vertical reinforcement ratio of wall M2 was higher than the proposed minimum. 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 or a smaller end zone length, as shown in Table 3. Wall M4 had a similar end zone vertical reinforcement ratio as wall M1, but used 2 D16 mm reinforcement bars instead of 4 D10 bars to investigate using larger diameter bars right at the wall end. All of the walls were designed to meet the revised NZS 3101 provisions for transverse reinforcement in both the compression toes and the central web region of the wall.

The phase II wall tests are current in progress. Secondary cracks are expected to occur in these modified walls with vertical reinforcement strains distributed evenly within the plastic hinge region. Buckling of vertical reinforcement should be delayed as the concentration of strains at large cracks should be avoided. Further modelling research will focus on the seismic behaviour of walls designed in accordance with minimum vertical reinforcement worldwide.

Table 3 Details of four test walls in Phase II

Wall	Vertical reinforcement ratio (%)			Vertical reinforcement		End zone length (mm)	Horizontal reinforcement ratio (%)	End region ties (mm)	Cross ties (mm)
	End zone	Web	Total	End zone	Web				
M1	1.00	0.465	0.673	4@10	10@10	210	0.25	R6@60	R6@150
M2	1.44	0.465	0.804	4@12	10@10	210	0.25	R6@60	R6@150
M3	0.72 (1.0)	0.465	0.589	2@12	10@10	210 (150)	0.25	R6@60	R6@150
M4	1.28	0.465	0.757	2@16	10@10	210	0.25	R6@60	R6@150

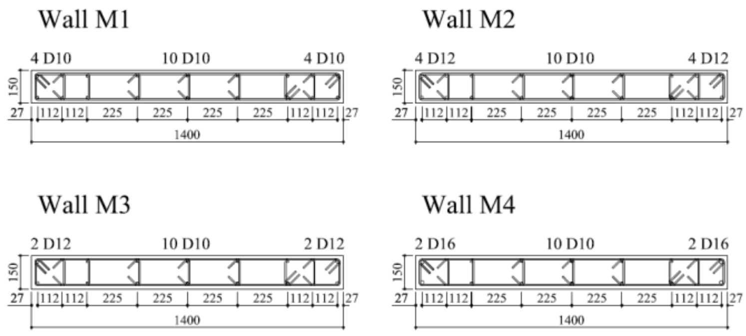


Figure 5. Cross sections of test wall specimens

CONCLUSIONS

Six RC walls designed in accordance with the current minimum vertical reinforcement requirements in NZS 3101:2006 (A2) were tested. The experimental results showed that the current minimum vertical reinforcing requirements in NZS 3101:2006 are insufficient to ensure that a large number of well distributed secondary cracks form in the plastic hinge region. Results from additional detailed finite element analyses indicated that wall size, reinforcement type and concrete strength had a significant effect on the cracking behaviour and lateral drift capacity of these lightly reinforced concrete walls. A second series of experimental test consisting of four RC walls designed according to NZS 3101:2006 (A3) is underway to verify and refine the proposed minimum vertical reinforcement requirements.

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