

# Experimental validation of minimum vertical reinforcement requirements for ductile concrete walls

Yiqiu Lu, Ronald J. Gultom, Quincy Q. Ma, Richard S. Henry

**Yiqiu Lu** is a research fellow in the Department of Civil and Environmental Engineering at University of Auckland, New Zealand. His research interests include the seismic design of reinforced concrete structures, precast concrete construction and low-damage seismic design.

**Ronald Gultom** holds a doctor of philosophy in structural engineering from University of Auckland, New Zealand, where his research focus was on developing techniques to overcome inaccuracies in hybrid seismic simulation. He is now a specialist engineer in the investigation of concrete structures.

**Quincy Ma** is a senior lecturer in the Department of Civil and Environmental Engineering at University of Auckland, New Zealand. He is a structural dynamic and structural mechanics specialist, and an experimentalist with a research focus on earthquake engineering.

ACI member **Richard Henry** is a senior lecturer in the Department of Civil and Environmental Engineering at University of Auckland, New Zealand. His research interests include the seismic design of reinforced and prestressed concrete structures, precast concrete construction, and low-damage seismic design. Rick sits on the technical committee of the New Zealand Concrete Structures Standard NZS 3101.

## Abstract

Recent experimental and numerical research has confirmed that reinforced concrete (RC) walls with distributed minimum vertical reinforcement in accordance with existing design standard provisions are unlikely to form a large number of distributed cracks and are only suitable for walls designed for low ductility demands. As a result, the minimum vertical reinforcement limits for ductile walls in the New Zealand Concrete Structures Standard (NZS 3101:2006) were recently amended to require additional vertical reinforcement to ensure well distributed secondary cracking. A series of four large-scale RC walls were tested to investigate the seismic performance of RC walls with

additional reinforcement at the end regions of the wall. The test walls were designed to represent flexural dominant RC walls in multi-storey buildings with reinforcement ratio, bar diameter and number of reinforcement bars in the end zone of the wall varied. The experimental results confirmed that 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 previously tested walls with minimum distributed reinforcement. The increased secondary cracks greatly increased the spread of plasticity and helped to delay reinforcement buckling. Furthermore, the additional vertical reinforcement limits proposed for the end region of ductile walls were found to be appropriate at the threshold at which well distributed secondary cracks form. It is recommended that similar vertical reinforcement limits be adopted in ACI 318 and other concrete design standards.

**Key words:** reinforced concrete; wall; seismic design; minimum vertical reinforcement; plastic hinge region; reinforcement buckling; reinforcement fracture; concrete design standards.

## INTRODUCTION

To achieve high ductility capacity during earthquakes, reinforced concrete (RC) walls should be designed to form well distributed flexural cracks in the plastic hinge region so that the vertical reinforcement yields over a significant length. In contrast to this, several lightly reinforced concrete (RC) walls in multi-storey buildings were observed to have formed only a limited number of cracks in the plastic hinge region during the 2010/2011 Canterbury earthquakes in New Zealand<sup>1,2</sup>. In response to these observations, researchers questioned whether the current minimum vertical reinforcements for RC walls were sufficient to generate a large number of cracks at plastic hinge region<sup>3</sup>. Lu et al.<sup>4</sup> 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). The test results showed that RC walls designed with minimum 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

1 cracks at the wall base. Primary cracks occur as a result of the flexural cracking strength of the wall  
2 being exceeded, whereas secondary cracks occur based on the local tensile stresses induced by the  
3 reinforcement into the surrounding concrete. In addition to fracture of vertical reinforcement at  
4 wide cracks, the concentrated cracking behaviour can cause additional problems such as wall  
5 sliding, early reinforcement buckling, and large axial elongations<sup>4, 5</sup>.

6 Lu and Henry<sup>6, 7</sup> developed a detailed finite element model for lightly reinforced concrete walls and  
7 conducted a series of numerical analyses to evaluate and compare the cracking and lateral load  
8 behaviour of RC walls with minimum vertical reinforcement in accordance with different concrete  
9 design standards, including ACI 318, CSA-A23.3, Eurocode 8, GB50010 and NZS 3101<sup>8-12</sup>. The  
10 numerical analysis results confirmed that the RC walls with a fixed distributed minimum  
11 reinforcement ratio of 0.25% or less (as per ACI 318-14) could not generate a large number of  
12 distributed cracks in the wall plastic hinge, resulting in premature reinforcement fracture and low  
13 drift capacities. It was also found that requiring additional reinforcement at ends of the wall, as is  
14 the case for several design standards, could improve the cracking behaviour and ductility.

15 Based on these findings and the observation in the 2010/2011 Canterbury earthquakes, new  
16 amendments have been proposed to the minimum vertical reinforcement limits in NZS 3101:2006  
17 (Amendment 3)<sup>9</sup> for ductile walls. The proposed amendments require additional vertical  
18 reinforcement to be placed in the ends of the wall to ensure that well distributed secondary cracking  
19 in ductile walls. The proposed requirements also account for concrete and reinforcement strengths  
20 and limit the ratio between end zone reinforcement and web reinforcement.

21 To experimentally examine the seismic response of walls with additional vertical reinforcement  
22 placed in the ends of the wall, a second series of tests were conducted on four RC walls. The four  
23 test walls were designed to investigate the vertical reinforcement ratio at the ends of the wall,  
24 reinforcing bar diameter, and number of reinforcing bars in walls with identical axial loads and  
25 material properties. The experimental results including crack pattern, failure mode and overall  
26 hysteric response are presented and data from a detailed array of instrumentation are discussed in

terms of crack distribution, deformation components, curvature distribution, plastic hinge length, vertical reinforcement strain and reinforcement buckling.

## RESEARCH SIGNIFICANCE

Recent research indicated that the distributed minimum vertical reinforcement requirements in current concrete design standards (including ACI 318-14) are insufficient to ensure that well distributed cracking occurs in the plastic hinge region of ductile RC walls. An experimental investigation was conducted to investigate the seismic response of walls with additional vertical reinforcement at the ends of the wall. The test results confirmed the amount of vertical reinforcement required at the ends of the wall to achieve ductile behaviour and can be used to justify amendments to minimum vertical reinforcement requirements for ductile RC walls that resist seismic actions in concrete design standards.

## PROPOSED MINIMUM REINFORCEMENT

Amendments proposed to NZS 3101:2006 (Amendment 3) resulted in an increase in the minimum required vertical reinforcement in limited ductile and ductile plastic regions of RC walls. The results of numerical analysis<sup>7</sup> showed that the new NZS 3101 provisions were most suitable for ductile RC walls when compared to the requirements in other concrete design standards. The proposed requirement was sufficient to ensure that well distributed cracks formed in the wall plastic hinge, resulting in large deformation capacity prior to reinforcement fracture.

The proposed minimum vertical reinforcement requirements are illustrated in Fig 1. In addition to minimum distributed vertical reinforcement ( $\rho_1$ ), additional vertical reinforcement is required in the end zone of the wall ( $\rho_{1e}$ ) which extends for a length of  $0.15l_w$  from extreme tension fibre of the wall. In the central region of the wall, the required minimum distributed vertical reinforcement ratio in NZS 3101:2006 (Amendment 3) can be expressed as Eq. 1, which is identical to the equation previously used for the minimum total vertical reinforcement ratio in NZS 3101:2006 (Amendment 2).

$$\rho_l \geq \frac{\sqrt{f'_c}}{4f_y} \text{ (MPa) or } \geq \frac{3\sqrt{f'_c}}{f_y} \text{ (psi)} \quad (1)$$

In the two end zones, the required reinforcement ratio has been doubled, as shown by Eq. 2

$$\rho_{le} \geq \frac{\sqrt{f'_c}}{2f_y} \text{ (MPa) or } \geq \frac{6\sqrt{f'_c}}{f_y} \text{ (psi)} \quad (2)$$

Eq. 2 was developed to ensure that well distributed secondary cracks formed at the tension edge of the wall, and was derived by assuming that the yield strength of end zone reinforcement is greater than the tensile capacity of the corresponding concrete section, as shown by Eq. 3, where  $A_{ct}$  is the area of concrete in the end zone,  $f_{ct}$  is the mean axial tensile strength,  $A_s$  is the area of end zone vertical reinforcement and  $f_y$  the yield strength of the vertical reinforcement.

$$A_s f_y \geq A_{ct} f_{ct} \quad (3)$$

Assuming that the area of concrete approximately equals the gross area of the section, the required end zone reinforcement ratio was calculated by Eq. 4, where the mean axial tensile strength  $f_{ct}$  was estimated as  $0.52\sqrt{f'_c}$  (MPa, or  $6\sqrt{f'_c}$  in kpi).

$$\rho_{le} \geq \frac{A_s}{A_{ct}} = \frac{f_{ct}}{f_y} = \frac{0.52\sqrt{f'_c}}{f_y} \text{ (MPa) or } \geq \frac{6\sqrt{f'_c}}{f_y} \text{ (psi)} \quad (4)$$

The minimum end zone reinforcement requirement in Eq. 4 is a function of the 28-day specified concrete strength and specified reinforcing steel yield strength. However, in reality the mean strength is higher than the lower characteristic specific strength and could be further increased by long-term strength effect and dynamic strength enhancement<sup>13, 14</sup>. Therefore, the committee also considered the effects of dynamic loading, drying shrinkage of concrete and average long-term material strengths. Details of the factors considering the parameters are shown in Eq. 5 and included:

- 1.2 multiplier on  $f_{ct}$  for the increase in concrete tensile strength due to dynamic loading rates;
- 0.85 multiplier on  $f_{ct}$  to allow for tensile strength reduction due to drying shrinkage;

- 1.2 multiplier on  $f'_c$  to represent the mean target compressive strength relative to the specified strength (5th percentile);
- 1.1 multiplier on  $f'_c$  for the increase in concrete compressive strength due to age;
- 1.1 multiplier on  $f_y$  for the increase in steel yield strength due to dynamic loading rates;
- 1.08 multiplier on  $f_y$  to represent the mean strength of reinforcement relative to the lower-characteristic strength (5th percentile).

$$\rho_{le} \geq \begin{cases} \frac{1.2 \times 0.85 \times 0.52 \sqrt{1.2 \times 1.1 f'_c}}{1.1 \times 1.08 f_y} = \frac{\sqrt{f'_c}}{2 f_y} & (\text{MPa}) \\ \frac{1.2 \times 0.85 \times 6 \times \sqrt{1.2 \times 1.1 f'_c}}{1.1 \times 1.08 f_y} = \frac{6 \sqrt{f'_c}}{f_y} & (\text{psi}) \end{cases} \quad (5)$$

To ensure that a reasonable degree of distribution of reinforcement occurs, there is also a requirement that the reinforcement ratio in the central web region shall not be less than 30% of that in the end zone of the wall. This requirement is to allow the secondary cracks to propagate sufficiently from the end zone to the central region of the wall. If the content of vertical reinforcement in the end zone ( $0.15l_w$ ) is significantly greater than the content of distributed vertical reinforcement in the central region, the wall can be vulnerable to widely spaced cracks in the central region, causing premature of web reinforcement fracture and large shear deformations<sup>2, 7, 15</sup>.

## EXPERIMENTAL INVESTIGATION

The experimental program comprised of four large-scale rectangular RC walls that were subjected to pseudo-static cyclic loading. The wall specimens were comparable to the six walls with distributed reinforcement that were previously tested by Lu et al.<sup>4</sup>, with the key parameter investigated being the concentrated reinforcement at the ends of the wall. The test setup, loading protocol, instrumentation and material properties of the four current tests were all consistent with the previous tests<sup>4</sup>.

### Test walls

A summary of the main parameters for the four test walls is shown in Table 1 and drawings of the

wall specimens are shown in Fig 2. All the test walls had a length of 1.4 m (55.1 in), a height of 2.8 m (110.2 in) and a thickness of 150 mm (5.9 in), which were identical with the six walls previously tested by Lu et al.<sup>4</sup>. The shear span ratio was 4 and the axial load ratio was 3.5% for all test walls, which were consistent with wall C6 from the previous tests. The axial load ratio applied to the test walls represents the most common axial ratio for this type of RC wall in New Zealand, and corresponded to an actual axial load of 294 kN (66.1 kips)<sup>4</sup> during the top of the wall during testing. The test walls were designed to represent a 40-50% scale version of multi-storey flexure-dominant RC walls (approximately 16-25 m, 4-8 stories) with ductile detailing requirements in accordance with NZS 3101:2006 (Amendment 3)<sup>9</sup>.

The vertical reinforcement was designed using G300E reinforcement (300 MPa specified yield strength) and a specified concrete strength ( $f'_c$ ) was 30 MPa (4351 psi), resulting in a minimum required vertical reinforcement ratio in the ends of the wall and central web region of the wall of 0.91% and 0.46% using Eq. 1 and Eq. 2, respectively, where the wall end zone length was  $0.15l_w$  or 210 mm (8.3 in). The distributed reinforcement ratio for all the four walls was 0.47%, resulting two layers of five D10 (deformed G300E, diameter = 10 mm or 0.39 in) bars placed at 225 mm (8.9 in) centers over the central wall web region. The end zone vertical reinforcement ratio of the four test walls was varied from 0.72% to 1.44%. Wall M1 was designed to closely satisfy the proposed minimum vertical reinforcement requirements (Eq. 4), which had an end zone reinforcement ratio of 1.0%, resulting in four D10 bars placed at the ends of the wall. Wall M2 used four D12 (deformed G300E, diameter = 12 mm or 0.47 in) bars at the ends of the wall, resulting in an end zone vertical reinforcement ratio of 1.44%, which was higher than the proposed requirements. Wall M3 did not satisfy the proposed minimum vertical reinforcement requirements at the ends of the wall with just two D12 bars at each end of the wall, and was designed to investigate either a reduced end zone reinforcement ratio (0.72%) or a smaller end zone length (150 mm or 5.9 in), as shown in Fig 2. Wall M4 had a similar end zone vertical reinforcement ratio as wall M1, but used two D16 (deformed G300E, diameter = 16 mm or 0.63 in) reinforcing bars instead of four D10 bars to

investigate using larger diameter bars right at the ends of the wall. For the comparison wall C6 designed as per NZS 3101:2006 (Amendment 2), two layers of seven D10 bars were evenly distributed at 225 mm (8.9 in) centers over the wall length with no additional vertical reinforcement placed in the ends of the wall, resulting in a total minimum vertical reinforcement ratio of 0.53%.

It should be noted that if same materials are used, the distributed minimum vertical reinforcement ratio of 0.47% required by NZS 3101:2006 (Amendment 3) was larger than the 0.25% fixed limit required by ACI 318-14<sup>11</sup> and CSA A23.3-14<sup>10</sup> and 0.2% required by Eurocode 8<sup>16</sup>. In addition, the end zone reinforcement ratio of 1.0% was similar to the limit in CSA A23.3-14<sup>10</sup> and GB 50010-2010<sup>12</sup> but larger than the 0.5% fixed limit in Eurocode 8<sup>16</sup>.

The horizontal reinforcement was also designed in accordance with NZS 3101:2006. Only minimum horizontal reinforcement was required to satisfy shear capacity, resulting in R6 (plain G300E, diameter = 6 mm or 0.24 in) stirrups distributed evenly at 150 mm (5.9 in) centers over the wall height, as shown in Fig 2. The shear demand to capacity ratio, defined as the shear at nominal flexural strength ( $V_{mn}$ ) divided by the nominal shear strength ( $V_n$ ) calculated in accordance with NZS 3101:2006 and ACI 318-14 (identical) are also provided in Table 1. The shear demand to capacity ratio was significantly less than 1 for all test walls, which indicated that the walls were likely to be flexure dominant. According to NZS 3101:2006 (Amendment 3)<sup>8</sup>, anti-buckling ties are required within the compression region for ductile walls and the spacing of the ties should be less than  $6d_b$ . To meet this ductility requirement, R6 stirrups were placed in the wall toes at 60 mm (2.4 in) centers over the lower 1.4 m (55.1 in) of the wall height for all the test walls, as shown in Fig 2. Cross ties were also placed between the vertical reinforcement in the wall web region at 150 mm (5.9 in) centers over the lower 1.4 m (55.1 in) of the wall height, as was also proposed in recent amendments. Wall C6 had identical horizontal reinforcement and anti-buckling ties with the four test walls while no cross ties placed in the wall web<sup>4</sup>.

## Test setup

The test setup shown in Fig 3 was designed to simulate the expected seismic loading on the lower



portion of the scaled RC wall that represented designs appropriate for tall buildings and was similar with that used for previous tests by Lu et al.<sup>4</sup>. The two vertical actuators were programmed to apply an axial load and a moment that was calculated based on the real-time measurement of the force in the horizontal actuator to achieve a constant axial load ratio and shear span ratio during tests<sup>17</sup>.

Additional details of the actuator setup, foundations, out-of-plane restraint, anchorage of vertical reinforcement and grouting between wall panel and foundation were described by Lu et al.<sup>4, 17</sup>.

### **Loading protocol**

The loading protocol applied to the test walls was developed in accordance with ACI 374.2R-13<sup>18</sup> and ACI ITG-5.1-07<sup>19</sup> and also consistent with previous tests by Lu et al.<sup>4</sup>. The axial load was applied prior to cyclic lateral load and kept constant throughout the testing. A combination of force-control and drift-control was used during applying the cyclic lateral load. The lateral drifts during drift-controlled cycles were 0.2%, 0.25%, 0.35%, 0.5%, 0.75%, 1.0%, 1.5%, 2.0%, 2.5% and 3.5% with three cycles at each drift level. Drift to west was defined to be positive and to the east as negative, where the directions are defined in Fig 3.

### **Instrumentation**

The instrumentation of the tests is shown in Fig 4. The forces and displacements applied by each actuator were monitored using internal load cells and LVDTs. A total of 9 displacement gauges were placed at each edge up the height of the wall to monitor axial strains and curvatures. Shear deformations in the wall were measured using displacement gauges in “X” configurations over two panel regions. In addition, the average reinforcement strains were measured using external displacement gauges over a 150 mm gauge length. Strain penetration of the vertical reinforcement at the wall-foundation interface was measured using a displacement gauge connected to the bottom stud welded on the vertical reinforcing bar and foundation. Displacement gauges were also used to measure any potential vertical and horizontal slip at the wall-to-foundation, wall-to-loading beam, and foundation to strong-floor joints.

### **Material properties**

1 Grade 300E reinforcing steel produced by Pacific Steel Group in accordance with AS/NZS 4671<sup>20</sup>  
 2 was used for the test walls. Six samples of each type of reinforcing bar were tested to confirm their  
 3 stress-strain behaviour. The average ultimate strain  $\epsilon_u$  was determined as the uniform elongation  
 4 over a 100 mm (3.9 in) gauge length at maximum stress. The measured mechanical properties of R6  
 5 (plain G300E, diameter = 6 mm or 0.24 in), D10 (deformed G300E, diameter = 10 mm or 0.39 in),  
 6 D12 (deformed G300E, diameter = 12 mm or 0.47 in) and D16 (deformed G300E, diameter = 16  
 7 mm or 0.63 in) are listed in Table 2. The stress-strain relationships for all the four types of the  
 8 reinforcement used in the test walls had an initial linear-elastic response, a yield plateau, and a non-  
 9 linear strain hardening phase until rupture.

10 The measured mechanical properties of the concrete at the time of testing each of the four walls are  
 11 listed in Table 3. Six concrete cylinders with a diameter of 100 mm (3.9 in) and a height of 200 mm  
 12 (7.8 in) were made alongside each wall panel with three cylinders used for compression tests to  
 13 determine the measured compressive strength ( $f_{cm}$ ) and the other three used for split cylinder tests to  
 14 estimate the tensile strength ( $f_{ct}$ ). The tensile strength listed in Table 3 are average splitting tensile  
 15 strengths calculated directly from the test conducted in accordance with NZS 3112.2<sup>21</sup>. The  
 16 modulus of elasticity ( $E_c$ ) was determined as the secant stiffness from the origin to 50% of the peak  
 17 concrete compressive strength. To simulate the effect of average long-term average concrete  
 18 strengths, a 40 MPa (5801 psi) concrete was targeted for the test walls. This represented a specified  
 19 design strength of 30 MPa (4351 psi) multiplied by the long-term strength modification factors in  
 20 Eq. 5 ( $30 \times 1.2 \times 1.1 = 39.6$  MPa (5843 psi)). The measured concrete compressive strengths at the time  
 21 of testing were around 36.5 MPa (5294 psi), slightly lower than the target long-term strength.  
 22 Accordingly, the measured tensile strengths listed in Table 3 were about 2.80 MPa (406 psi), which  
 23 was also lower than the assumed average long-term tensile strength which was calculated to be  
 24 3.27 MPa (474 psi). However, the measured tensile strengths were close to the assumed tensile  
 25 strength of  $0.52\sqrt{f'_c}$ , which was calculated to be 2.85 MPa (413 psi) using the specified concrete  
 26 strength of 30 MPa (4351 psi).

## TEST OBSERVATIONS AND RESULTS

Table 4 provides a summary of the drift cycle during which key observations were made during the tests, including first cracking, concrete spalling, reinforcement buckling, core concrete crushing and reinforcement fracture. The test wall C6 tested by Lu et al.<sup>4</sup> is also included in Table 4 for comparison. An example of the evolution of the crack pattern and damage for the typical test wall M1 at key drift levels is shown in Fig 5. Additional photos for other test walls are published in Lu<sup>17</sup>. The crack patterns as well as the maximum measured crack widths at the end of the test of six test walls are shown in Fig 6. The crack pattern visible on both the north and south sides of the wall was similar, with minor differences as the cracks propagated through the wall. Therefore, the crack patterns in Fig 6 were drawn only from the southern side, where the left-hand end is west and right-hand end is east. Finally, the final condition and the moment-displacement hysteresis response for the four tested walls are shown in Fig 7 and Fig 8, respectively.

### Wall M1

Wall M1 was considered the baseline wall with an end zone vertical reinforcement ratio of 1.0%, which closely satisfied the minimum requirements proposed in NZS 3101:2006 (Amendment 3). The wall response was dominated by flexural behaviour with a large number of horizontal cracks extending over almost the entire wall height. In contrast to the comparable previously tested wall C6 that had minimum distributed reinforcement as per NZS 3101:2006 (Amendment 2) and were dominated by 3-4 large cracks, the cracks in wall M1 were more evenly distributed over the plastic hinge region. As shown in Fig 5 and Fig 6a, wall M1 had more cracks and a smaller crack spacing compared to that of wall C6. The maximum crack width at a drift of 2.5% was around 7 mm (0.28 in), which was significantly less than the 20 mm wide (0.79 in) crack width observed for wall C6 at the same drift level. Furthermore, unlike wall C6 where all the flexural cracks formed prior to 0.5% lateral drift, new secondary cracks continued to form in wall M1 during cycles up to  $\pm 1.5\%$  lateral drift. The cracking behaviour of wall M1 indicated that concentrating a greater portion of the reinforcement in the ends of the wall can significantly improve the crack distribution and control of

1 crack widths.

2 Concrete spalling and reinforcement buckling in wall M1 were delayed when compared to wall C6

3 due to the even distribution of plasticity. As shown in Table 4, the concrete at the corners in the wall

4 started to spall and buckling of the corner vertical reinforcement initiated during cycles to lateral

5 drifts of  $\pm 2.0\%$ , while reinforcement buckling of wall C6 occurred during cycles to lateral drifts of

6  $+1.5\%$ . Increasing the vertical reinforcement resulted in an increased number of secondary cracks,

7 which allowed the reinforcement strains to be more evenly distributed over the plastic hinge region.

8 This even distribution of plasticity helped delay buckling of the vertical reinforcement by avoiding

9 the large tensile strains that develop in vertical reinforcement at wide concentrated cracks, as was

10 the case for walls with minimum distributed vertical reinforcement described by Lu et al.<sup>4</sup>. The two

11 buckled reinforcing bars at the east end fractured during the second and third cycle to  $+2.5\%$  lateral

12 drift, respectively. At west end, one reinforcing bar fractured during the third cycle to  $-2.5\%$  lateral

13 drift. The delaying of reinforcement buckling significantly reduced the damage of wall M1 when

14 compared to wall C6 at the same drift level<sup>4</sup>, as shown in Fig 5. At  $2.5\%$  lateral drift, wall C6 was

15 significantly damaged and the strength dropped below 80% of the peak strength, while wall M1 was

16 still in an acceptable condition after reinforcement fracture with the largest crack still less than

17 10 mm (0.39 in) and so the wall was loaded until  $3.5\%$  lateral drift. Subsequent web reinforcing bar

18 buckling and fracture occurred during the cycles to  $\pm 3.5\%$  lateral drift. The final condition of the

19 west and east end of the wall is shown in Fig 7a.

20 As observed in the force-displacement response in Fig 8a, the initial cross section stiffness of wall

21 M1 was slightly lower than expected due to unexpected cracking before the test when the vertical

22 actuators were installed. However, the inelastic response was not affected and was stable up until

23  $2.0\%$  lateral drift when buckling of the vertical reinforcement caused strength degradation on

24 subsequent cycles. The test wall achieved a peak strength of 594.3 kN-m (438.4 kips-ft) and -

25 601.4 kN-m (443.6 kips-ft) at  $+1.5\%$  and  $-1.5\%$  lateral drift, respectively. A drop of 20% of the peak

26 strength occurred when the buckled reinforcing bar fractured during the third cycle to  $+2.5\%$  lateral

1 drift.

## 2 **Wall M2**

3 Wall M2 was identical to wall M1 except that D12 bars replaced the D10 bars in the ends of the  
4 wall, resulting in a larger end zone reinforcement ratio of 1.44%. Similar to wall M1, the behaviour  
5 of wall M2 was dominated by flexure with a large number of cracks occurring over the full wall  
6 height. As shown in Fig 6b, more obvious inclined shear cracks were observed in the central web  
7 region of the wall and more secondary cracks occurred at ends of the wall due to the larger end zone  
8 reinforcement ratio. New secondary cracks continued to form up until drift cycles of  $\pm 1.5\%$ . In the  
9 later stages of the test, the inclined web cracks were wider than the cracks at wall edge due to the  
10 difference in the distributed reinforcement ratio in wall web and reinforcement ratio at the ends of  
11 the wall.

12 Concrete spalling and reinforcement buckling was delayed in wall M2 when compared to wall M1.  
13 As shown in Table 4, concrete spalling and reinforcement buckling initiated in wall M2 during  
14 cycles to  $\pm 2.0\%$  lateral drift and  $\pm 2.5\%$  lateral drifts, respectively. It appears that the stability of the  
15 larger diameter reinforcing bars in wall M2 provided increased stability and delayed the initiation of  
16 reinforcement buckling. At east end, the buckled reinforcing bar fractured during the first cycle to  
17  $+3.5\%$  lateral drift. Subsequent reinforcing bars in the end zone fractured during the next two cycles  
18 to  $+3.5\%$  lateral drift. At the west end, severe buckling of the outmost vertical reinforcement caused  
19 localized lateral instability to initiate during the first cycle to  $+3.5\%$  lateral drift. The wall edge  
20 moved about 30 mm (1.18 in) sideways to north (instrumentation face) at the height of 400 mm  
21 (15.7 in) above the foundation. The out-of-plane movement extended over half of the wall length.  
22 Vertical reinforcement fracture did not occur at the east end until the third cycle to  $-3.5\%$  drift. The  
23 final condition at the east end of wall M2 is shown Fig 7b.

24 The first flexural crack initiated during the first cycle to  $+0.2\%$  lateral drift with a wall base  
25 moment of 294.6 kN-m (217.3 kips-ft), or roughly 43% of the peak strength. As shown in Fig 8b,  
26 the inelastic response of wall M2 was stable up until  $\pm 2.5\%$  lateral drift when buckling of the

vertical reinforcement occurred causing a gradual degradation in wall strength. The test wall reached a peak strength of 668.6 kN-m (493.2 kip-s-ft) and -631.3 kN-m (465.7 kip-s-ft) at +2.5% and -2.0% lateral drift, respectively, slightly larger than that of wall M1 due to the increase of the end zone reinforcement ratio. Three vertical reinforcing bars fractured during the second cycle to +3.5% lateral drift, leading a 20% drop of peak strength. The moment-displacement hysteresis curves was slightly larger than that of wall M1 due to the higher reinforcement ratio, indicating that more energy was dissipated during the test.

### Wall M3

Wall M3 was comparable to wall M1 and M2 except for a smaller end zone reinforcement ratio of 0.72% corresponding to two D12 reinforcing bars (or alternatively a 1.0% end zone reinforcement ratio with a smaller end zone length of 150 mm or 5.9 in). Similar to wall M1 the behaviour of wall M3 was also dominated by flexure with a large number of cracks occurring over the full wall height. Wall M3 had slightly less secondary cracks than that in wall M1 due to the reduced end zone reinforcement, as shown in Fig 6c. The maximum crack width was also larger than that of wall M1 and M2, with the largest crack opening to 8 mm (0.31 in) wide. When compared to wall C6, the cracking behaviour was still significantly improved, indicating that even a small increase in reinforcement at the ends of the wall can improve the cracking behaviour. New secondary cracks continued to form up until drift cycles to  $\pm 1.0\%$ , which was also earlier than wall M1 and M2.

Concrete spalling occurred during the first cycle to  $\pm 2.0\%$  lateral drift, which was similar with that of wall M1. However, reinforcement buckling was delayed in wall M3 when compared to wall M1 even though the reinforcement ratio of wall M3 was less than that of wall M1, and initiated during the first cycle to  $\pm 2.5\%$  lateral drift. This again indicated that the stability of the larger diameter reinforcing bar (D12 compared to D10) can help delay reinforcement buckling. At the east end of the wall, the two buckled reinforcing bars fractured during the first cycle to +3.5% lateral drift. At the west end, the two buckled reinforcing bars fractured during the second cycle to -3.5% lateral drift. Subsequent web reinforcing bars buckled and fractured during the third cycles to  $\pm 3.5\%$

lateral drift. The final condition at the west and east end of wall M3 is shown in Fig 7c. The first flexural crack initiated during the first cycle to +0.2% lateral drift with a wall base moment of 294.5 kN-m (217.2 kips-ft), or roughly 52% of the peak strength. The inelastic response of wall M3 was stable up until  $\pm 2.5\%$  lateral drift when buckling of the vertical reinforcement occurred and caused a gradual degradation in wall strength, as shown in Fig 8c. Wall M3 achieved a peak strength of 560.8 kN-m (413.6 kips-ft) and -532.0 kN-m (392.4 kips-ft) at +1.5% and -2.0% lateral drift, respectively, smaller than that of wall M1 due to the decrease of the end zone reinforcement ratio. Two vertical reinforcing bars fractured during the second cycle to -2.5% lateral drift, leading a 20% drop of peak strength.

#### Wall M4

Wall M4 had a similar end zone reinforcement ratio as wall M1 but used two D16 reinforcing bars instead of four D10 reinforcing bars. The crack pattern of wall M4 was similar to that observed for wall M1, and was dominated by a significant number of flexural cracks over the full wall height, as shown in Fig 6d. As the end zone reinforcement was more concentrated in the ends of the wall, cracks near wall edge were slightly denser than that in wall M1. In addition, new secondary cracks also formed up until drift cycles to  $\pm 1.5\%$ .

As previously discussed, due to the stability of the larger diameter reinforcing bars, concrete spalling and buckling of vertical reinforcement in wall M4 were both delayed when compared to wall M1. Concrete spalling was observed during cycles to  $\pm 2.0\%$  lateral drift and vertical reinforcement buckling occurred during the cycles to  $\pm 2.5\%$  lateral drift. During the third cycles to  $\pm 2.5\%$  lateral drift, reinforcement buckling became more severe and the core concrete started to crush at both ends of the wall. Two vertical reinforcing bars at the west end of the wall fractured during the first cycle to -3.5% lateral drift and one vertical reinforcing bar at the east end of the wall fractured during the third cycle to +3.5% lateral drift. The final condition on the west and east end of wall M4 is shown in Fig 7d.

A few flexural cracks initiated simultaneously during the first cycle to +0.2% lateral drift with a

wall base moment of 306.9 kN-m (226.4 kips-ft), or roughly 48% of the peak strength. As shown in Fig 8d, the inelastic response of wall M4 was more stable than that of wall M1 after  $\pm 2.0\%$  lateral drift as reinforcement buckling was delayed in wall M4 test. The peak strengths of wall M4 were 630.2 kN-m (464.8 kips-ft) and -630.7 kN-m (465.2 kips-ft) at +1.5% and -1.5% lateral drift, respectively, slightly larger than that of wall M1 but similar to that of M2. Strength degradation occurred after  $\pm 2.5\%$  lateral drift due to reinforcement buckling, and the strength dropped below 80% of the peak strength when reinforcement fractured during the first cycle to -3.5% lateral drift. The moment-displacement hysteresis curves were slightly larger than that of wall M1 due to the higher reinforcement ratio, indicating that more energy was dissipated during the test.

## DISCUSSION OF TEST RESULTS

The instrumentation used for the test walls allowed for both the global and local behaviour of the wall to be investigated. The test results are interpreted in the following sections in terms of cracking distribution, deformation components, curvature distribution, plastic hinge length, reinforcement strains and reinforcement buckling. Sensor data is plotted and discussed up until 2.5% lateral drift as displacement gauges at the low part of the wall were compromised during cycles to 3.5% lateral drift due to reinforcing bar buckling and fracture.

### Crack distribution

The maximum measured crack widths and the average crack spacing observed during the first cycle to 2.5% lateral drift for each of the four test walls are plotted against the end zone vertical reinforcement ratio in Fig 9 alongside comparable walls C2 and C6 that also had a shear span ratio of 4 but had distributed minimum vertical reinforcement. The average crack spacing was estimated as the height over which the cracking extended up the wall divided by the number of the cracks observed at the wall edge. As shown in Fig 9, the maximum crack widths at 2.5% drift for the four test walls were less than 8 mm (0.31 in), significantly smaller than the 20 mm (0.79 in) wide cracks that were measured for both walls C2 and C6. As end zone reinforcement ratio increased, the maximum crack width and average crack spacing both decreased, indicating that a greater number



1 of closely spaced secondary cracks occurred in the walls with a higher end zone reinforcement ratio.  
2 It should be noted that this trend was more obvious when the reinforcement ratio was small. For  
3 example, the maximum crack width decreased from 20 mm (0.79 in) to 8 mm (0.31 in) and the  
4 average crack spacing decreased from 275 mm (10.8 in) to 139 mm (5.5 in) when the end zone  
5 reinforcement ratio increased from 0.5% to 0.72%. However, when the reinforcement ratio was  
6 larger, the trend started to flatten off as an optimum number of secondary cracks had formed.  
7 Comparing the four test walls M1 to M4 for which the end zone reinforcement ratio ranged from  
8 0.72% to 1.44%, the maximum crack width varied from 4 mm (0.16 in) to 8 mm (0.31 in) and the  
9 average crack spacing varied from 93 mm (3.7 in) to 139 mm (5.5 in). This indicated that lumping  
10 vertical reinforcement in the ends of the wall can significantly improve the cracking behaviour of  
11 lightly reinforced concrete walls that might otherwise be controlled by discrete widely spaced  
12 cracks. However, when the cracking behaviour of an RC wall is controlled by well distributed  
13 cracks, further increases to reinforcement in the ends of the wall will not have a significant effect on  
14 the crack distribution. This trend was also observed in the numerical models conducted by Lu and  
15 Henry<sup>7</sup>.

16 Based on the discussion above, it appears that the end zone reinforcement ratio in wall M3 was at  
17 the threshold for ensuring that well distributed secondary cracks initiate. However, as previously  
18 discussed, the measured concrete tensile strength of wall M3 was lower than the assumed average  
19 long-term tensile strength and can only represented the tensile strength when using specified  
20 concrete strength. If the concrete strength gained the assumed long-term concrete strength or higher,  
21 the ratio between the tensile capacity of reinforcement and concrete would decrease and so  
22 secondary cracks would likely be reduced, resulting in poor seismic performance. Furthermore,  
23 during the test, wall M3 was subjected to pseudo-static cyclic loading and the wall was observed to  
24 have reasonable cracking behaviour. However, if wall M3 was subjected to dynamic loading that  
25 might increase the concrete tensile strength, secondary cracks would be further reduced. For walls  
26 M1, M2 and M4 that had larger end zone reinforcement ratios, there was a reasonable margin of

1 safety to ensure secondary cracks. The end zone reinforcement ratio in wall M1 that was in-line  
2 with the proposed amendments in NZS 3101:2006 (Amendment 3) can be considered as an  
3 appropriate threshold to ensure that well distributed secondary cracks form when the wall is  
4 subjected to dynamic earthquake loading. A reduced end zone reinforcement ratio or a smaller end  
5 zone length may result in undesirable cracking behaviour during earthquakes, while a larger end  
6 zone reinforcement ratio does not provide a noticeable improvement in the crack distribution. Given  
7 that the requirements in ACI 318-14 and Eurocode 8 typically result in less vertical reinforcement  
8 than that required by NZS 3101:2006 (Amendment 3), the behaviour of comparable walls designed  
9 in accordance with other design standards may result in significant less secondary cracking. The  
10 behaviour of walls designed as per CSA A23.3-14 which require similar end region reinforcement  
11 ratio but significantly less distributed reinforcement ratio by NZS 3101:2006 (Amendment 3) also  
12 needs to be further assessed.

### 13 **Deformation components**

14 As with previous tests<sup>4</sup>, the measurement of the wall flexural deformations were split into four  
15 panel regions (F1, F2, F3 and F4), as shown in Fig 4, to quantitatively compare the deformation  
16 contribution up the wall height. The flexural deformations were calculated by double-integrating the  
17 curvatures calculated from the vertical displacement gauges located along both wall edges,  
18 assuming plane sections remain plane. The shear deformations were computed directly from the  
19 diagonal displacement gauges in accordance with the methods proposed by Hiraishi<sup>22</sup>. The strain  
20 penetration was difficult to quantify because it could not be easily separated from the wide flexural  
21 crack at the wall base for lightly reinforced concrete walls. Therefore, the lateral displacement  
22 resulting from reinforcement strain penetration into the foundation was not calculated separately  
23 and was instead included in the flexural component F1. The contributions of the flexural and shear  
24 deformation components to the total wall displacement during the first cycle to each lateral drift  
25 level for each test wall as well as wall C6 are shown in Fig 10. The summation of these five  
26 deformation components correlated well with the lateral displacement measured directly at the top

1 of the wall, with an error typically less than 10%. The components were slightly unsymmetric for  
2 some walls due to differences in the crack distribution between the east and west ends of the wall.

3 For all the four test walls, the flexural displacements were considerably larger than the shear  
4 displacements, which roughly accounted for 9-12% of the total lateral displacement. The  
5 contribution of shear deformation in the four walls was similar with that recorded in other ductile  
6 RC wall tests, such as walls tested by Dazio et al.<sup>23</sup> (shear deformation around 10%). However, the  
7 shear deformations were larger than that observed in wall C6 that had only minimum distributed  
8 vertical reinforcement as per NZ S3101:2006 (Amendment 2), which were extremely flexure  
9 dominant with shear deformations that contributed to less than 5% of the total lateral displacement.

10 The difference of the flexural deformation components between the four test walls and wall C6 was  
11 also substantial. For wall C6, components F1 and F2 that accounted for 18% of the wall height  
12 contributed nearly 75% of the total lateral displacement in different drift levels. These local  
13 deformations confirmed that inelastic deformations were not distributed over a large length of the  
14 wall height, and that the wall behaviour was instead dominated by 2-3 main flexural cracks at the  
15 wall base. As the cracks in walls M1 to M4 were more evenly distributed over the plastic hinge  
16 region, the flexural deformation of the four test walls were also more distributed among the flexural  
17 panel regions up the wall height. Unlike wall C6, components F1 and F2 roughly contributed 50%  
18 of the total lateral displacement for test walls M1 to M4. In addition, component F4 contributed 5-  
19 10% of the total deformation for all four test walls while that region contributed almost nothing to  
20 the response of wall C6. This confirmed that inelastic deformation of the four test walls was  
21 distributed over a larger length of the wall height. Furthermore, for wall C6, the relative  
22 contribution of F1 increased sharply and F2 decreased sharply between drifts from -2.0% to -2.5%  
23 drift. The concentration occurred as the dominant flexural crack at the wall base widened at large  
24 drifts, resulting in large inelastic strains and fracture of the vertical reinforcement. At the same drift  
25 level, this localisation behaviour was not observed in the four test walls. The relative contribution of  
26 each component of the four test walls kept similar as lateral drift increased. In addition, the

contribution of deformation components of all the four test walls were similar, indicating end region vertical reinforcement ratio ranging from 0.72% to 1.44% did not significantly influence the deformation components.

#### **Curvature distribution**

The average curvatures up the wall height, calculated from the displacement gauges at the wall edge, at the first cycle to each drift target for each test wall as well as wall C6 are shown in Fig 11.

The curvature distributions further confirm the observed wall behaviour and correlate well with the crack patterns shown in Fig 6 and the deformation component shown in Fig 10. For wall C6, the curvature distribution contained a few sharp peaks at the location of wide cracks as opposed to continuously distributed curvatures over the wall height. However, for walls M1 to M4, the curvatures were more continuously distributed over the wall height, as shown in Fig 11. The characteristic of this curvature distribution was also found in other ductile wall tests<sup>23</sup> and was typical of ductile RC members with a well distributed plastic hinge<sup>24</sup>. The curvature distribution of wall M2 with the largest end zone reinforcement ratio were slightly more even and stable than that of other walls. For walls M3 and M4, there were a few locations where curvature was not distributed linearly and was attributed to the randomness of the crack pattern and was also observed by other ductile wall tests<sup>25</sup>.

#### **Plastic hinge length**

The plastic hinge length ( $l_p$ ) calculated at each drift cycle for each of the four test walls as well as walls C2 and C6 are plotted alongside the theoretical plastic hinge length calculated in accordance with NZS 3101:2006 in Fig 12. The measured plastic hinge length was calculated using the same methods in previous tests<sup>4</sup> in accordance with Eq.6, where  $\varphi_m$  is the maximum curvature measured during the test,  $\varphi_y$  is the yield curvature defined as  $2\varepsilon_y/l_w$ <sup>26</sup> and  $\theta_p$  is the plastic rotation calculated by integrating the plastic curvature profile over the entire wall height.

$$\theta_p = (\varphi_m - \varphi_y)l_p \quad (6)$$

In NZS 3101:2006, plastic hinge length is calculated as the smaller of  $0.15M/V$  and  $0.5l_w$ , which is consistent with recommendations from previous researchers<sup>27, 28</sup>. The NZS 3101 plastic hinge length for these walls with a shear span ratio of 4 was controlled by  $0.5l_w$  (or 700 mm). The measured plastic hinge lengths in the positive and negative loading directions were not perfectly symmetric due to the influence of different crack patterns at two ends. As shown in Fig 12 the average plastic hinge lengths of wall C6 and wall C2 were about 450 mm and 400 mm, respectively, well below the theoretical plastic hinge length due to the concentrated inelastic behaviour at the wall base. However, for the tested walls M1 to M4, the average plastic hinge length calculated from the test response was roughly equal to 725 mm and correlated well with the plastic hinge length assumed by NZS 3101:2006. These results highlight that traditional assumptions for plastic hinge length analysis are not suitable for lightly reinforced walls with only minimum distributed vertical reinforcement, but are applicable for the walls with additional vertical reinforcement at the ends of the wall, as per the proposed requirements in NZS 3101:2006 (Amendment 3).

### **Vertical reinforcement strains**

The average tensile strains measured along the outmost vertical reinforcement up the height of the wall are plotted in Fig 13 for each test wall as well as wall C6. The strains were obtained by dividing the readings from the displacement gauges welded onto the vertical reinforcement by the gauge length of 150 mm. Strains measurements were compromised after the reinforcement buckled and so these values are not plotted in Fig 13. For wall M1 at west end and wall M2 at both ends, the bottom studs welded on the reinforcing bar were broken before test so the bottom reinforcement strains were plotted relative to the foundation instead. The strain distributions were different for drifts to west and east as the average strain measurements were affected by the crack distribution and the strain profiles are also dependent on the gauge length. The reinforcement strains in wall C6 was inconsistent up the wall height with inelastic strains concentrated at crack locations. However, for the four test walls with additional vertical reinforcement in ends of the wall, the reinforcement

1 strains were more evenly distributed over the plastic hinge region, which were similar with well  
2 detailed ductile RC walls<sup>23, 25</sup>. This finding indicated that the increased secondary cracking in the  
3 four test walls was sufficient to ensure that reinforcement strains were evenly distributed. The  
4 reinforcement strain distributions for test walls M1 to M4 were similar, indicating that increasing  
5 the reinforcement at the ends of the wall from 0.72% to 1.44% did not have a significant effect on  
6 the reinforcement strains.

### 7 **Reinforcement buckling**

8 As with the six test walls<sup>4</sup> with minimum distributed vertical reinforcement, the failure of all four  
9 test walls was also controlled by buckling and subsequent fracture of the vertical reinforcement. As  
10 presented by Lu et al.<sup>4</sup>, reinforcement buckling in the previous six test walls was attributed to the  
11 large crack widths and concentrated inelastic reinforcement strains. To compare the initiation of  
12 reinforcement buckling between the previous six test walls with distributed vertical reinforcement  
13 (C1-C6) and the current four test walls with additional reinforcement concentrated at the ends of the  
14 wall (M1-M4), the measured average reinforcement tensile strains and crack widths at the location  
15 of buckling in the cycle prior to reinforcement buckling for all the test walls are summarised in  
16 Table 5. The onset of reinforcement buckling was defined as visible distress during the test,  
17 typically in the form of the concrete spalling or vertical cracks initiating adjacent to the buckled  
18 reinforcement. It also should be noted that the reinforcement strains listed in Table 5 were average  
19 strains calculated by dividing the readings from the displacement gauges welded onto the vertical  
20 reinforcement by the gauge length of 150 mm.

21 In general, the reinforcement buckling was delayed in walls M1-M4, as discussed previously. The  
22 even distribution of plasticity helped to delay buckling of the vertical reinforcement by avoiding the  
23 large vertical reinforcement tensile strains that develop at wide discrete cracks. As shown in Table  
24 5, the average strain prior to reinforcement buckling for walls C1-C6 ranged from 2.2% to 4.5%  
25 with an average of 3.5% and 3.3% for the east and west ends of the wall respectively. For walls M1-  
26 M4, the average strain measured prior to reinforcement buckling were larger, ranging from 3.2% to

6.4% with an average of 4.4% and 4.7% for the east and west ends of the wall respectively. In contrast to the average reinforcement strains, the average crack width at the location of buckling for walls C1-C6 was about 6.5 mm, larger than the 4.0 mm average crack width for walls M1-M4. The larger average tensile strain and smaller crack widths indicated that more secondary cracks occurred in the walls with additional reinforcement concentrate at the ends of the wall. For the walls with discrete cracking behaviour, as was the case for walls C1-C6, the peak strain in the reinforcement at the concentrated crack is significantly higher than the average strain over a specific gauge length. However, for the walls with well distributed secondary cracks, as was the case for walls M1-M4, the reinforcement strains at cracks are close to the average strain over the gauge length. Therefore, larger average reinforcement tensile strain was achieved prior to buckling for walls M1-M4, which resulted in delaying reinforcement buckling

## CONCLUSIONS

Test results were presented for four RC walls that were designed with additional reinforcement concentrated at the ends of the wall in accordance with amendments to the minimum vertical reinforcement requirements for ductile walls in NZS 3101:2006 (Amendment 3). The test observations and results were compared with previously tested walls<sup>4</sup> that had only minimum distributed vertical reinforcement as per NZS3101:2006 (Amendment 2). The main conclusions drawn from this experimental study are summarized as follows:

- The four test walls were controlled by a large number of primary and secondary cracks over the wall height. The walls with a larger end zone vertical reinforcement ratio had more secondary cracks. However, when the end zone vertical reinforcement ratio was similar, the diameter and number of reinforcing bars in ends of the wall did not have a significant effect on cracking behaviour.
- The curvature and reinforcement strain distributions in the plastic hinge region of the four test walls were continuously distributed over the wall height. Increasing the end zone reinforcement ratio from 1.0% to 1.44%, and increasing the reinforcing bar diameter, did not

significantly influence the curvatures and reinforcement strain profiles. Additionally, typical plastic hinge length assumptions that are used to estimate drift capacity and curvature demands are suitable for the four test walls.

- The failure for all the four test walls was controlled by vertical reinforcement buckling and subsequent reinforcement fracture. When compared to test walls with minimum distributed vertical reinforcement, the increased vertical reinforcement content in the four test walls resulted in increased number of secondary cracks that allowed the reinforcement strains to be more evenly distributed over the plastic hinge region. The even distribution of reinforcement strains in the walls with additional vertical reinforcement at the ends of the wall meant that larger average reinforcement tensile strains could be achieved prior to reinforcement buckling when compared to the previously tested walls that had only distributed reinforcement. **In addition, the stability of the larger diameter reinforcing bars can help to delay the onset of reinforcement buckling for lightly reinforced concrete walls.**
- When compared to the minimum distributed vertical reinforcement requirements in NZS 3101:2006 (Amendment 2), the additional vertical reinforcement limits proposed for the end region of ductile walls in NZS 3101:2006 (Amendment 3) were found to be appropriate to ensure the secondary cracks occurred in the plastic hinge region. A reduced end zone reinforcement ratio or a smaller end zone length may result in undesirable cracking behaviour during earthquakes. Additionally, an end zone vertical reinforcement ratio larger than that proposed may result in slightly more secondary cracks, but no significant improvement in the seismic performance.
- It is recommended that concrete design standards adopt requirements for concentrating a greater portion of the reinforcement at the ends of RC walls to improve the cracking behaviour and ductility of walls with minimum vertical reinforcement. The minimum vertical reinforcement requirements for ductile RC walls in ACI, Eurocode 8, GB 50010-2010 and CSA A23.3-14 should be re-assessed to ensure that well distributed cracks develop



as intended in plastic hinge regions. The proposed requirements to calculate the end zone reinforcement ratio based on the long-term direct tensile strength of the concrete are considered to be appropriate to achieve this objective.

## **ACKNOWLEDGEMENTS**

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# FIGURES AND TABLES

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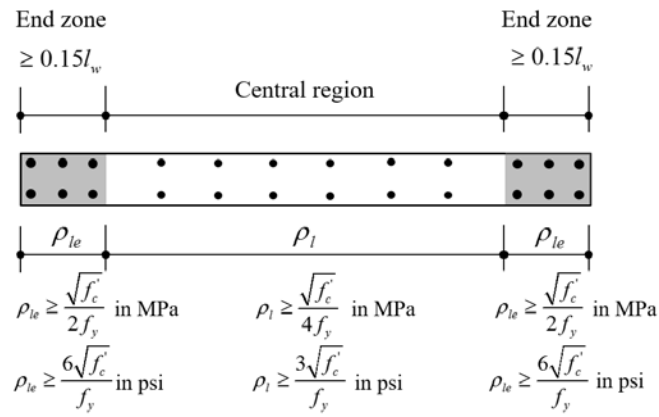
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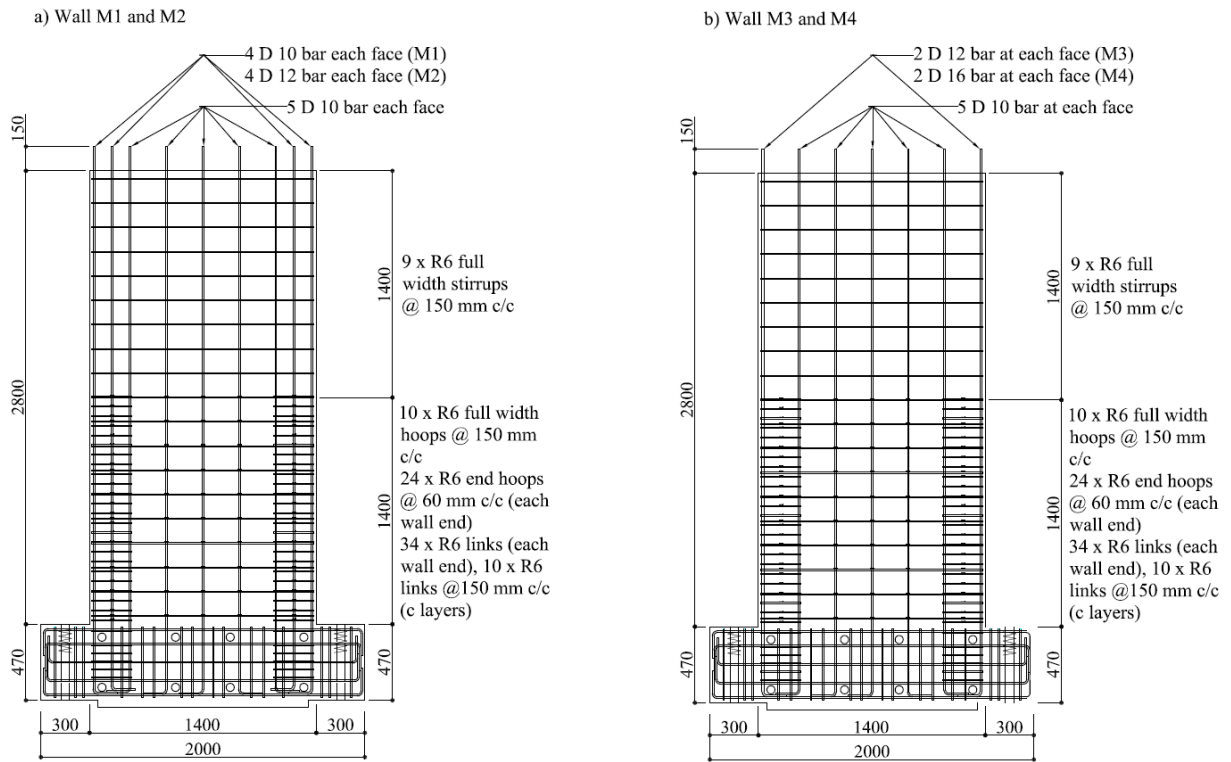
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*Fig 1 - New minimum vertical reinforcement for limited or fully ductile wall plastic regions in NZS 3101:2006 (Amendment 3)*

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(a) Elevations



(b) Cross sections

Fig 2 - Drawings of test wall specimens (D = deformed bar; R = round bar; units in mm; 1 mm = 0.039 in.)

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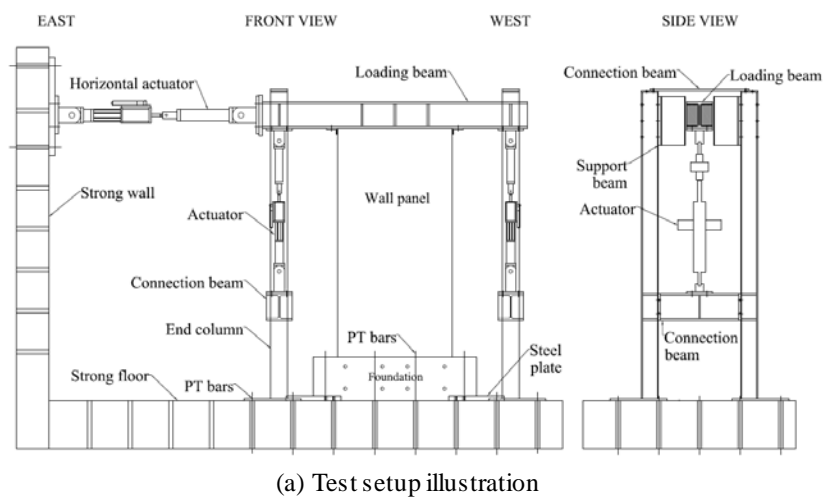
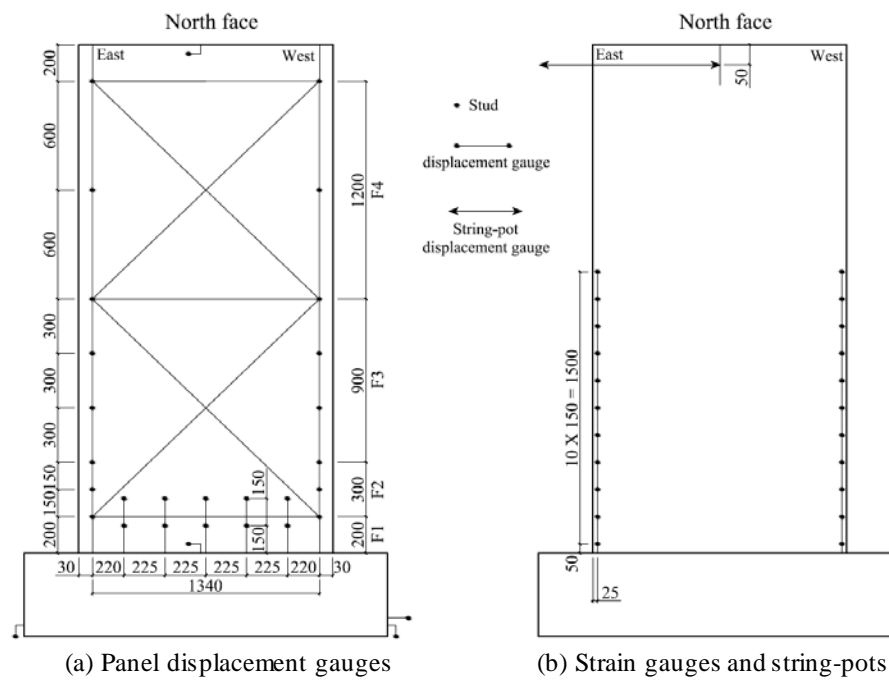


Fig 3 - Wall test setup

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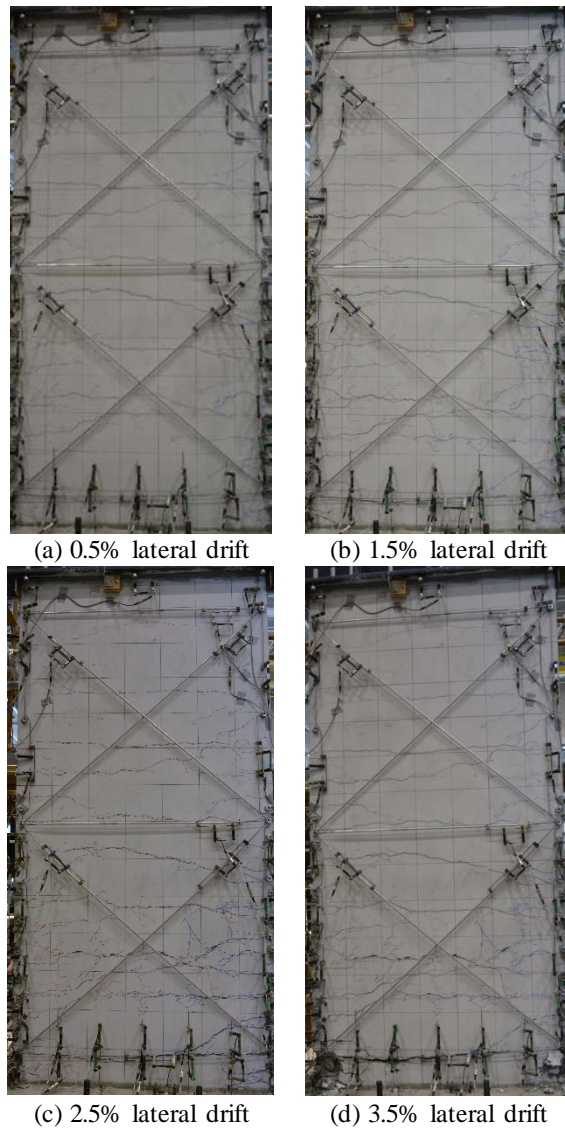


*Fig 4 - Instrumentation used for the test walls (units in mm; 1 mm = 0.039 in.)*

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*Fig 5 - Evolution of crack pattern and damage of wall M1*

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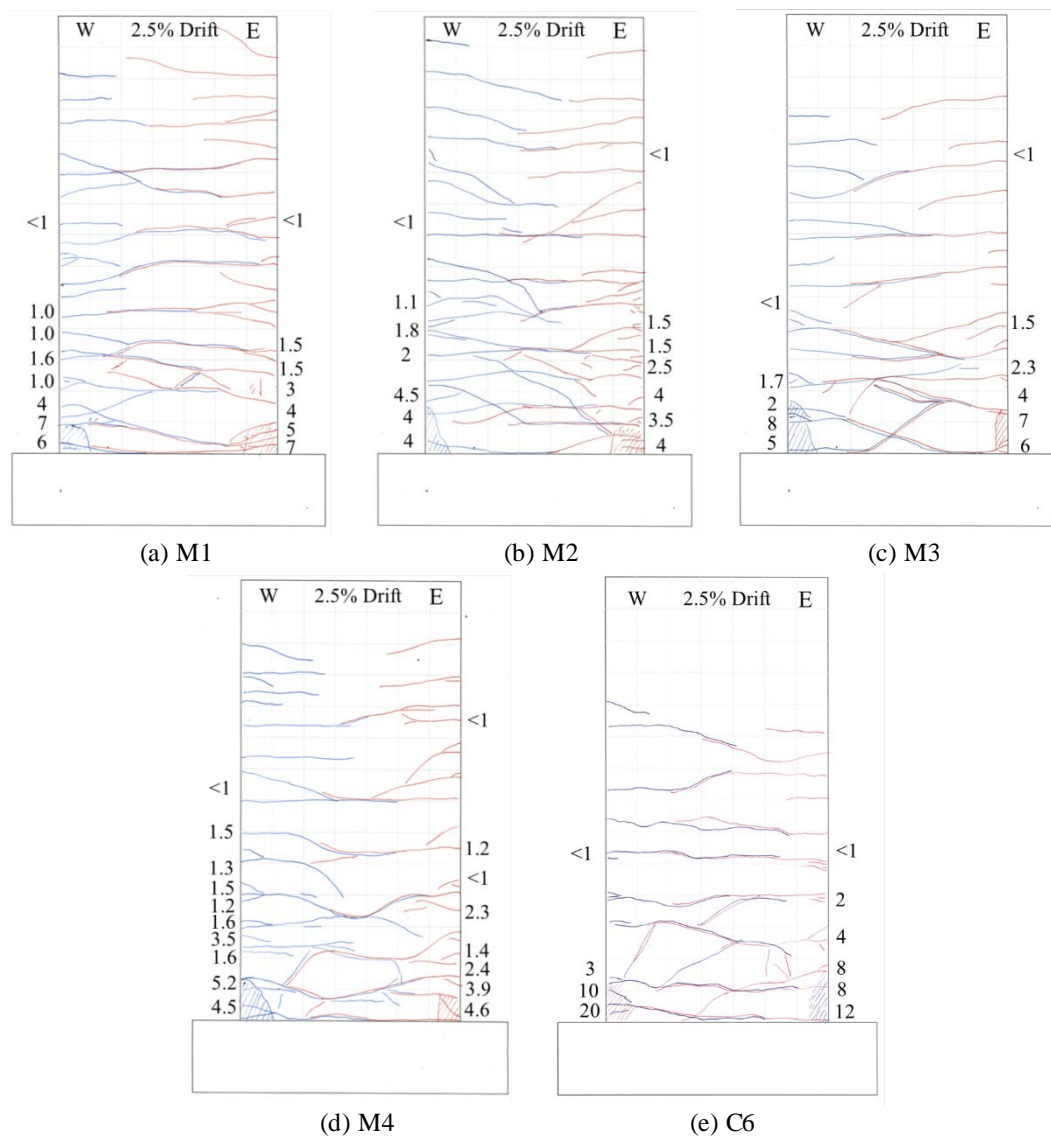
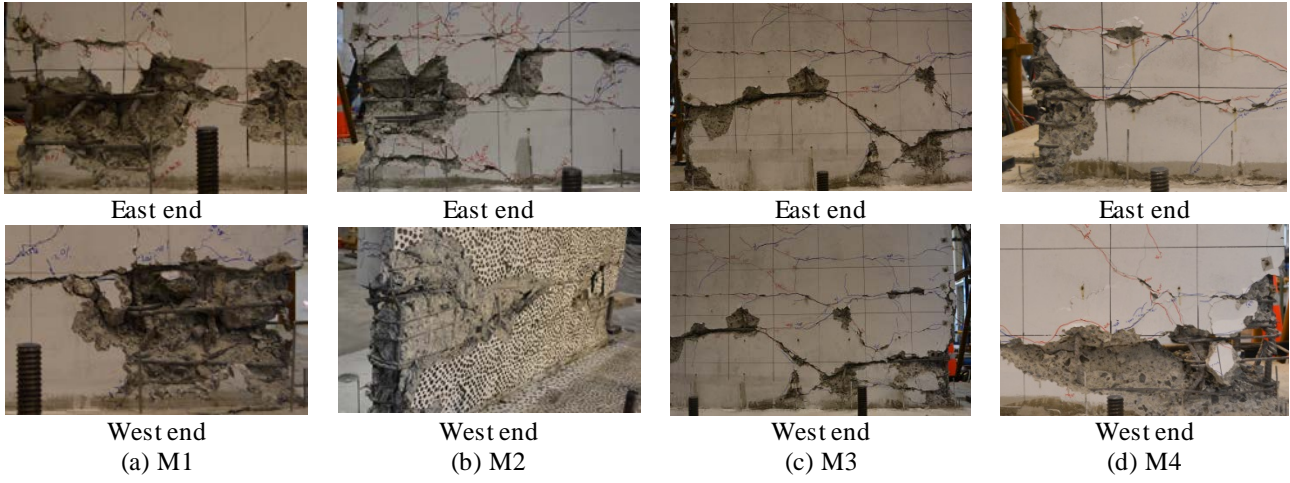


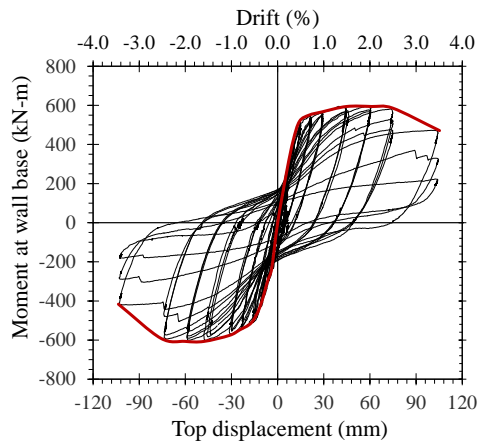
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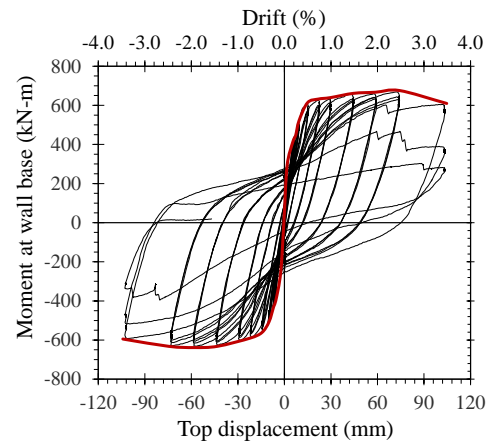
*Fig 7 - Photos of the wall toe at the end of each test*

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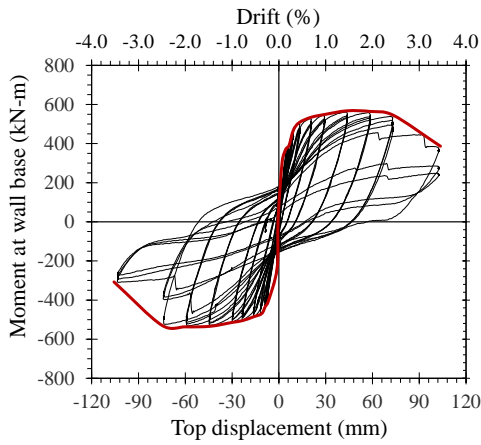
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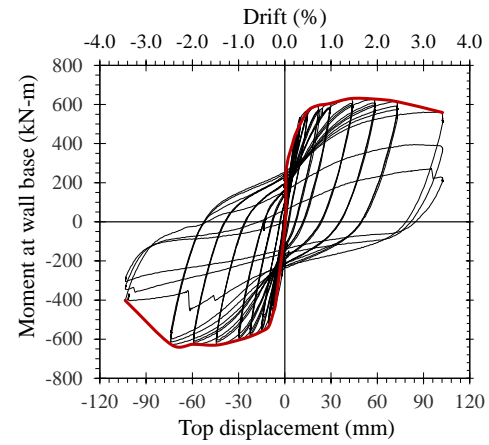
(a) Wall M1



(b) Wall M2

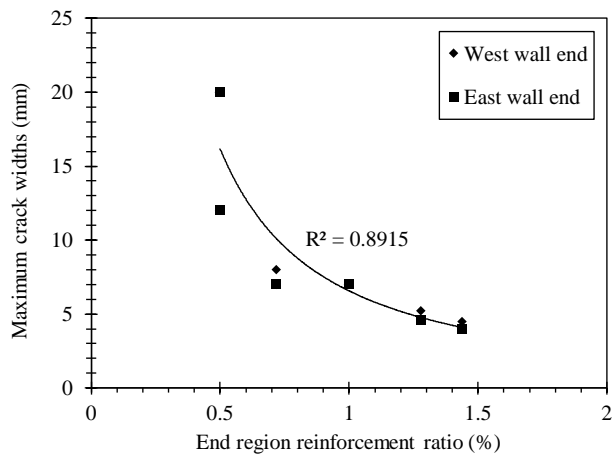


(c) Wall M3

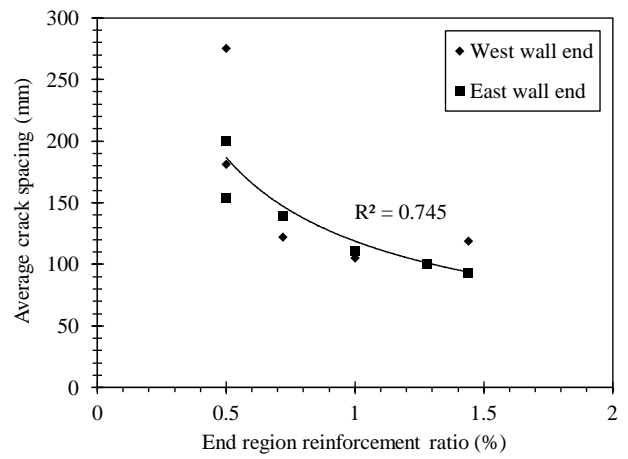


(d) Wall M4

*Fig 8 - Moment-displacement response for all five test walls (1mm = 0.039 in.)*



(a) Maximum crack width



(b) Average crack spacing

Fig 9 - Maximum crack width and average crack spacing at 2.5% lateral drift for test walls with a shear span ratio of 4

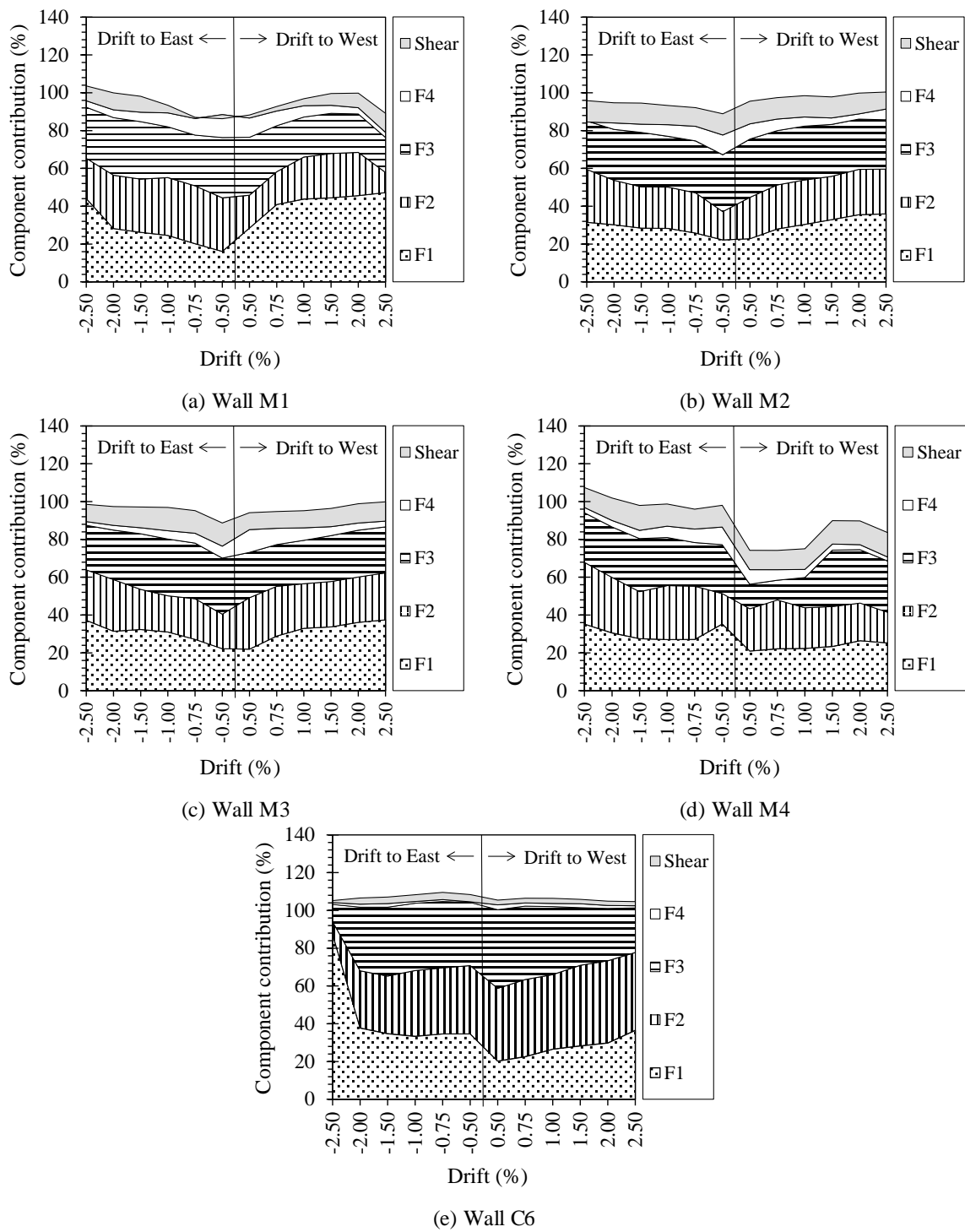
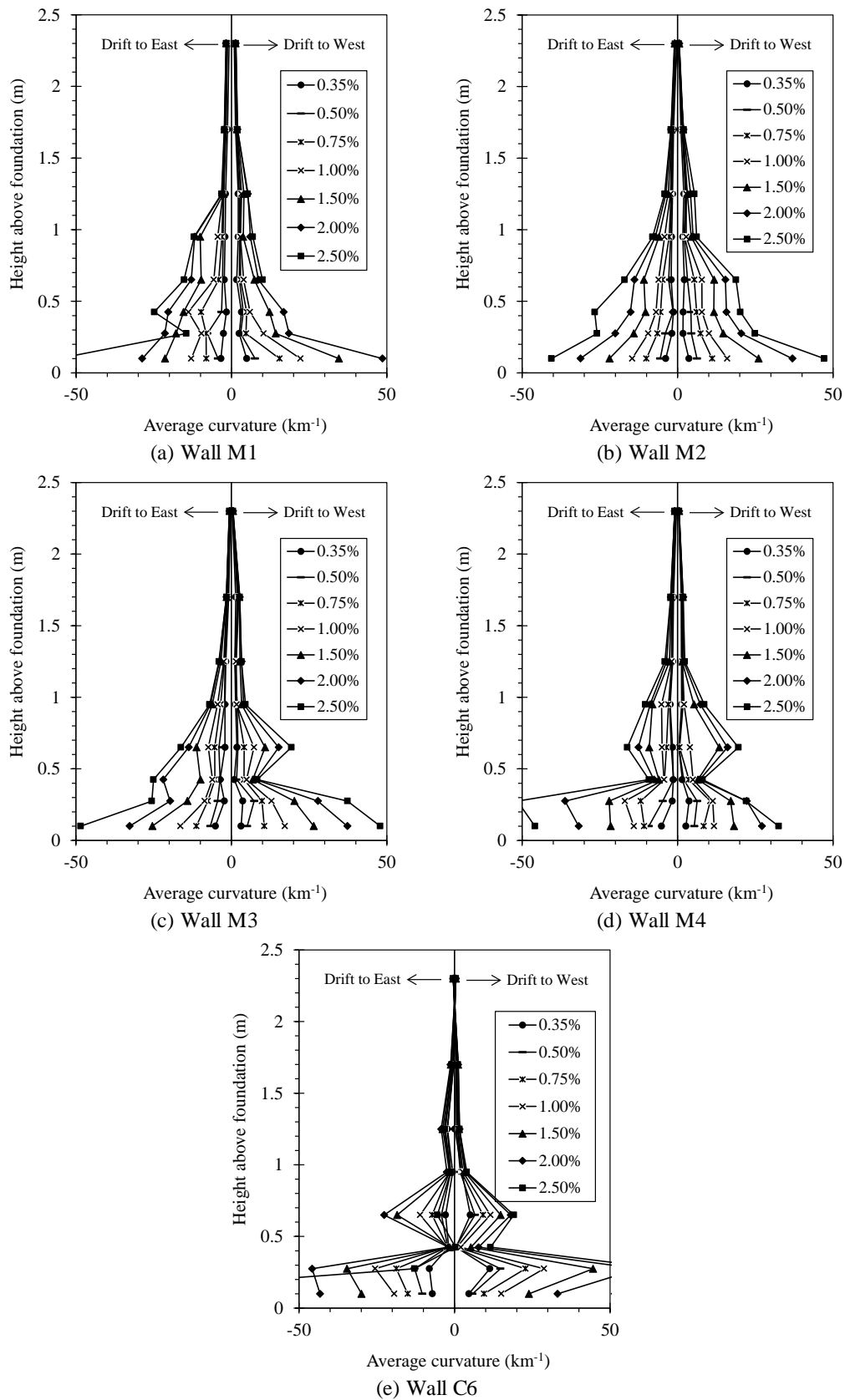
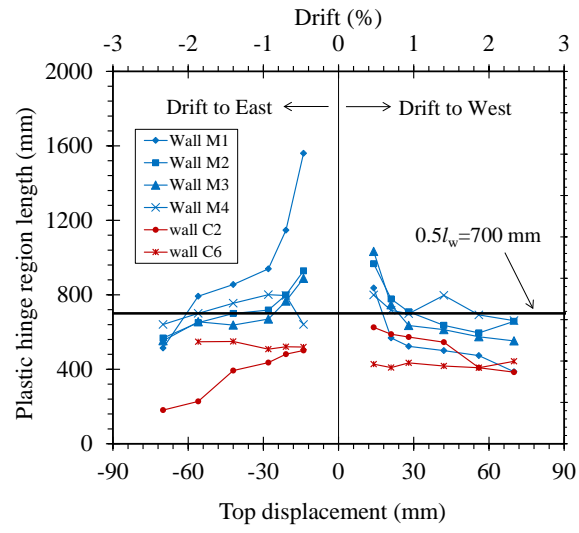


Fig 10 - Panel deformation components of the four test walls compared with wall C6



*Fig 11 - Curvature distributions over the height of the four test walls compared with wall C6 (1 mm = 0.039 in.)*

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Fig 12 - Calculated plastic hinge lengths of test walls with shear span ratio of 4 (1 mm = 0.039 in.)

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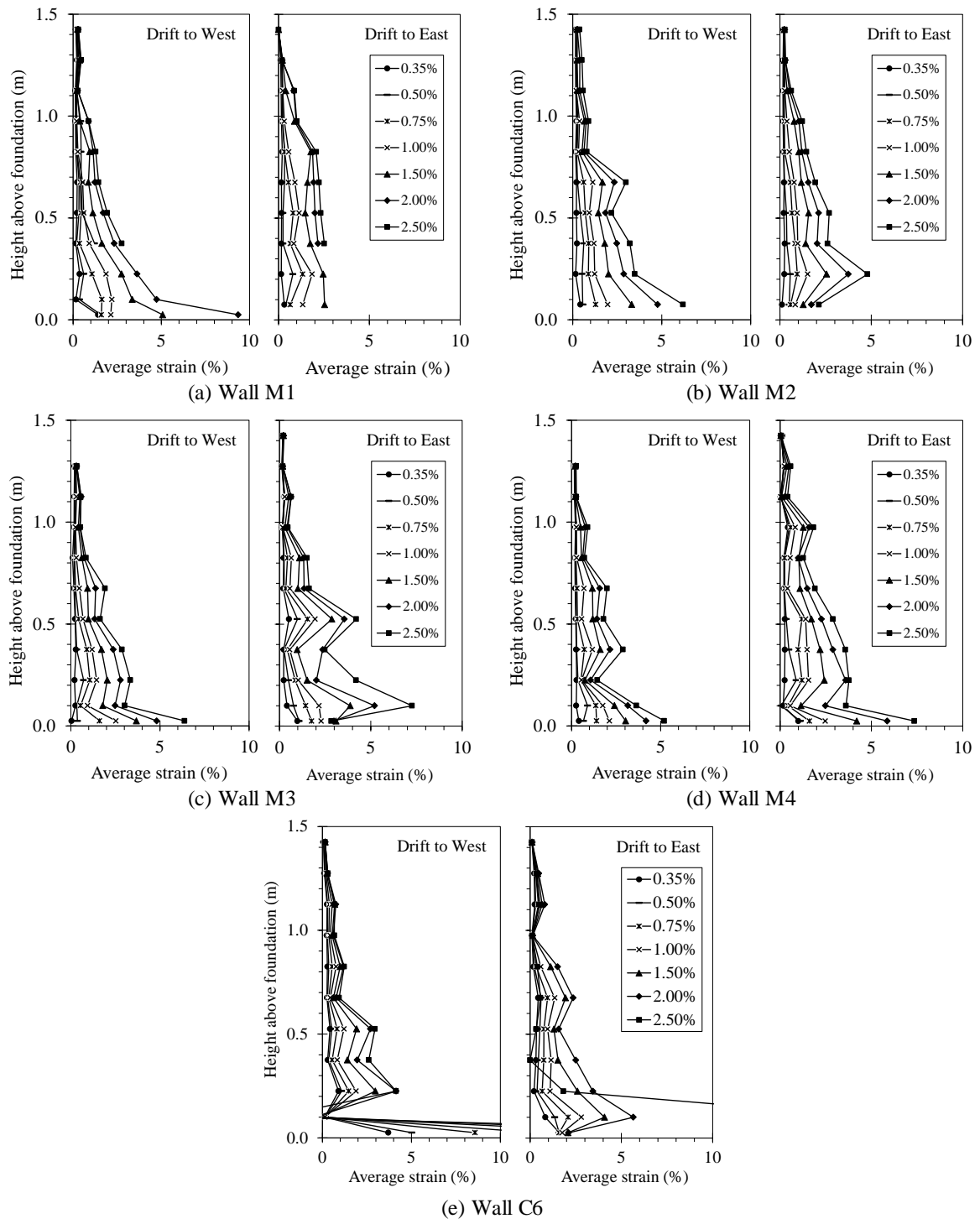


Fig 13 - Average strains along the vertical reinforcement of the four test walls compared with wall C6 (1 mm = 0.039 in.)



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Tables

2

Table 1 - Details of test walls (1 mm = 0.039 in.)

Wall	Shear span ratio	Axial load ratio	Shear demand to capacity ratio	Vertical reinforcement ratio (%)				End zone Reinforcement	End zone length (mm)	Horizontal reinforcement ratio (%)	End ties (mm)	Web ties (mm)
				End zone	Web region	Total	$\rho_l/\rho_{le}$					
M1	4	3.5%	0.26	1.00	0.47	0.67	0.47	4 D10	210	0.25	R6@60	R6@150
M2	4	3.5%	0.29	1.44	0.47	0.80	0.32	4 D12	210	0.25	R6@60	R6@150
M3	4	3.5%	0.25	0.72 (1.0)	0.47	0.59	0.65	2 D12	210 (150)	0.25	R6@60	R6@150
M4	4	3.5%	0.29	1.28	0.47	0.76	0.36	2 D16	210	0.25	R6@60	R6@150

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*Table 2 - Mechanical properties of reinforcing steel (D = deformed bar; R = round bar; 1 MPa = 0.145 ksi.)*

Reinforcement	$f_y$ (MPa)	$f_u$ (MPa)	$f_y/f_u$	$\epsilon_u$ (%)
R6	322	450	1.40	16.4
D10	387	484	1.25	13.2
D12	371	471	1.27	11.4
D16	334	467	1.40	14.5

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*Table 3 - Mechanical properties of concrete (1 MPa = 0.145 ksi; 1 GPa = 0.145 psi; 1 kg/m<sup>3</sup> = 0.062 lb/ft<sup>3</sup>)*

Test wall	$f_{cm}$ (MPa)	$E_c$ (GPa)	$f_{ct}$ (MPa)	$\rho_c$ (kg/m <sup>3</sup> )
M1	37.1	34.0	2.99	2402
M2	36.3	34.0	2.87	2409
M3	36.3	28.8	2.76	2419
M4	36.7	30.4	2.57	2352

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*Table 4 - Key observations for the four test walls compared with wall C6*

Test wall	Direction	First cracking	Concrete spalling	Reinforcement buckling	Core concrete crushing	Reinforcement fracture
M1	+	N/A	+2.0% <sup>1 a</sup>	+2.0% <sup>3</sup>	+3.5% <sup>3</sup>	+2.5% <sup>2</sup>
	-	N/A	-2.0% <sup>1</sup>	-2.0% <sup>2</sup>	-3.5% <sup>1</sup>	-2.5% <sup>3</sup>
M2	+	+0.2% <sup>1</sup>	+2.0% <sup>3</sup>	+3.5% <sup>1</sup>	+3.5% <sup>1 b</sup>	+3.5% <sup>1</sup>
	-	-0.2% <sup>1</sup>	-2.0% <sup>3</sup>	-2.5% <sup>2</sup>	-3.5% <sup>2</sup>	-3.5% <sup>3</sup>
M3	+	+0.2% <sup>1</sup>	+2.0% <sup>1</sup>	+2.5% <sup>1</sup>	+3.5% <sup>1</sup>	+3.5% <sup>1</sup>
	-	-0.2% <sup>1</sup>	-2.0% <sup>1</sup>	-2.5% <sup>1</sup>	-3.5% <sup>1</sup>	-2.5% <sup>2</sup>
M4	+	+0.2% <sup>1</sup>	+2.0% <sup>2</sup>	+2.5% <sup>1</sup>	+2.5% <sup>3</sup>	+3.5% <sup>3</sup>
	-	-0.2% <sup>1</sup>	-2.0% <sup>2</sup>	-2.5% <sup>3</sup>	-2.5% <sup>3</sup>	-3.5% <sup>1</sup>
C6	+	+0.12%	+1.0% <sup>3</sup>	+1.5% <sup>3</sup>	N/A	+2.5% <sup>3</sup>
	-	-0.12%	-2.0% <sup>1</sup>	-2.0% <sup>3</sup>	N/A	-2.0% <sup>2</sup>

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<sup>a</sup> superscript present the cycle number, <sup>b</sup> instability occurred

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*Table 5 - Average vertical reinforcement tensile strains and crack widths prior to reinforcement buckling (1 mm = 0.039 in)*

Test	Test wall	Drift at buckling (%)		Average reinforcement tensile strain (%)		Crack widths (mm)	
		Drift to west	Drift to east	Drift to west	Drift to east	Drift to west	Drift to east
First series <sup>4</sup>	C1	1.5	1.5	4.4	2.2	4	6
	C2	1.5	1.5	3.6	2.8	4	7
	C3	1.5	1.0	2.6	3.5	6	6.5
	C4	1.0	0.75	4.4	4.4	10	8
	C5	1.5	1.5	8.2*	2.9	6	4
	C6	1.5	2.0	4.1	4.1	7	6
Average				3.8	3.3	6.5	6.4
COV				0.20	0.25	0.39	0.27
Second series	M1	2.0	2.0	N/A**	4.1	3	3
	M2	3.5	2.5	4.8	3.5	5.5	4
	M3	2.5	2.5	5.2	6.4	5	6
	M4	2.5	2.5	3.2	5.2	4	4
Average				4.4	4.7	4.1	4.0
COV				0.20	0.24	0.28	0.31

\* buckling occurred in a different location where the measuring length is only 50 mm, other steel strains are based on gauge length of 150 mm. This value was not included when calculating the average value and the coefficient of variation; \*\* stud was broken prior to reinforcement buckling.

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