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Minimum Longitudinal Reinforcement Requirements for Boundary Elements of Limited Ductile Walls for AS 3600

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ABSTRACT: Observations of poor performance of reinforced concrete walls in recent earthquake events have been associated with a light amount of longitudinal reinforcement. In particular, single-crack failures have been observed for reinforced concrete walls that have an insufficient amount of longitudinal reinforcement to allow secondary cracking. It has been proposed that the next revision of the Concrete Structures code in Australia (AS 3600) increase the current minimum longitudinal reinforcement required for limited ductile reinforced concrete walls to mitigate against this type of failure in the event of a large earthquake. This research investigates the current and proposed longitudinal reinforcement requirements of AS 3600. A reinforced concrete wall is analysed using a state-of-the-art finite element modelling program for a range of different longitudinal reinforcement configurations. The wall detailed to the minimum longitudinal reinforcement requirements proposed for the next revision of AS 3600, which requires boundary elements, was found to allow secondary cracking and using less bars than a wall with distributed reinforcement. The displacement capacities of the walls that formed secondary cracks are found to be limited unless transverse reinforcement is used to confine the longitudinal reinforcement in the boundary ends of the wall.

Keywords: secondary cracking, unconfined, lumped, distributed, steel, concrete.

1 INTRODUCTION

In regions of low-to-moderate seismicity, such as Australia, the majority of the reinforced concrete (RC) walls are lightly reinforced (Wibowo *et al.*, 2013; Wilson *et al.*, 2015). Poor performance associated with the seismic performance of lightly reinforced walls has been observed in recent earthquake events (CERC, 2012; Wood *et al.*, 1991). Most notably, a single crack has been found to form in the plastic hinge zone of lightly reinforced walls leading to large strain concentrations in the reinforcement at this crack and potentially causing fracture of the reinforcement. For instance, it was likely that the RC core wall of the Pyne Gould Corporation building, which collapse in a non-ductile, brittle and catastrophic fashion during the February 22nd, 2011 Christchurch earthquake, had insufficient longitudinal reinforcement to transmit the required tension to initiate secondary cracking in the surrounding concrete (CERC, 2012). Thus, in comparison to the well distributed cracks (and corresponding distribution of strains) up a significant portion of the wall height as is usually observed experimentally, the yielding of reinforcement was 'confined to a short length resulting in a single wide crack in the potential plastic region at level 1' (CERC, 2012). Another lightly reinforced wall that was observed to have form a single crack at the base after the Christchurch event, with fractured longitudinal reinforcement crossing the crack, was located in the Gallery Apartments Building (CERC, 2012). Some subsequent studies confirmed that the wall had insufficient longitudinal reinforcement to allow secondary cracking (Henry, 2013; Henry *et al.*, 2014; Sritharan *et al.*, 2014).

The current minimum longitudinal reinforcement ratio (ρ_{wv}) required by the AS 3600:2009 (Standards Australia, 2009) Concrete Structures building code in Australia for RC walls is 0.15%. An investigation conducted by Hoult *et al.* (2017) focused on the minimum amount of longitudinal steel required for a RC wall to allow secondary cracking, and thus allowing a distribution of strains up the wall height from the base. Depending on the mechanical properties of the steel and strength of the concrete, Hoult *et al.* (2017) showed that a much higher ρ_{wv} than 0.15% was generally required to allow secondary cracking. An expression was derived from the study by Hoult *et al.* (2017) to determine the minimum longitudinal reinforcement ratio ($\rho_{wv.min}$) required to initiate secondary cracking in RC walls (Equation 1):

$$\rho_{wv.\min} = \frac{(t_w - n_t d_{bt}) f_{ct.fl}}{f_u t_w} \qquad (1)$$

where t_w is the thickness of the wall, n_t is the number of grids of transverse reinforcing bars, d_{bt} is the diameter of the transverse reinforcing bars, f_u is the ultimate tensile strength of the reinforcing steel and $f_{ct.fl}$ is the mean value of the flexure tensile strength of the concrete.

Some revisions for the design of reinforced concrete structures in Australia, and particularly for earthquake actions, are currently being proposed for the next revision of AS 3600. This includes increasing the minimum ρ_{wv} required by AS 3600 for 'limited ductile' RC walls. Limited ductile RC walls will require "boundary elements", which correspond to the ends of the wall to be detailed with a large amount of longitudinal reinforcement (in comparison to the web) and potentially an increase in transverse reinforcement ("confinement"). The recommended length of the boundary regions of the wall is $0.15L_w$, where L_w is the length of the wall. This length has been adopted from Eurocode 8 (EC-8) (CEN, 2004) and has subsequently been proposed as the boundary length of walls for NZS 3101:2006(A3), as discussed in Lu *et al.* (2016) and Cook *et al.* (2014). Equation 1 was used with conservative design values to result in a simplified expression (Equation 2) that has been proposed for the next revision of AS 3600 to calculate the required ρ_{wv} for the boundary ends of the wall. In deriving Equation 2, the t_w was assumed to be 300 mm with (n_t) 2 grids of transverse (d_{bt}) 16 mm reinforcement. The $f'_{ct.fl}$ was calculated using the equation given in AS 3600 (Standards Australia, 2009), while a factor of 1.32 was similarly used in Cook *et al.* (2014) to increase the concrete strength due to (i) aging and (ii) the ratio of mean to lower targeted characteristic strength. Furthermore, $1.08f_y$ is the lower characteristic ultimate strength of D500N reinforcing steel according to AS/NZS 4671:2001 (Standards Australia/New Zealand, 2001), while the 1.1 in the denominator of Equation 2 is due to the 'tensile strength increase of steel reinforcement due to dynamic loading' (Cook *et al.*, 2014). It should also be noted that the proposed revisions of the longitudinal reinforcement for limited ductile RC walls in AS 3600 will also require that the ρ_{wv} in the web of the wall be at least half of that calculated with Equation 2.

$$\rho_{wv} = \frac{(300 - (2 \times 16))(1.4 \times 0.6\sqrt{1.32\, f'_c}}{(1.1)(1.08 f_y)(300)} = \frac{0.7\sqrt{f'_c}}{f_y} \qquad (2)$$

where f'_c is the targeted 28-day compressive strength of concrete and f_y is the lower characteristic yield strength of the longitudinal reinforcing steel (e.g. 500 MPa for D500N bars).

Equation 2 has been recommended for AS 3600 and for the design of limited ductile RC walls to ensure secondary cracking will occur in the event of an earthquake. Secondary cracking is required to allow a distribution of tensile strains up the wall height, permitting the wall to deform as intended by the designer. Alternatively, if a ρ_{wv} less than that required by Equation 2 is used in the wall, it is likely that the longitudinal strains will be concentrated at a single, primary crack at the base of the wall, prohibiting the flexural deformation of the wall. However, Equation 1 (and subsequently Equation 2) was derived from a study by Hoult *et al.* (2017), which focused on RC walls with a *distributed* amount of longitudinal reinforcement, rather than *lumped* reinforcement in the boundary ends. Thus, a study is required to confirm that the expression that has been recommended for the minimum ρ_{wv} of limited ductile RC walls (Equation 2) in the next revision of AS 3600 allows secondary cracking and a distribution of longitudinal strains.

The aim of this paper is to investigate the displacement capacity and cracking distribution of a RC wall with different longitudinal reinforcement layouts. The layouts include different amounts of distributed and lumped longitudinal reinforcement that correspond to the current requirements of AS 3600:2009 (Standards Australia, 2009) and the requirements that are being proposed for the next revision of AS 3600. A state-of-the-art finite element modelling program is used for the numerical analyses. The results will ultimately indicate if the proposed requirements for the next revision of AS 3600 improve the performance and displacement capacity of RC structural walls in comparison to the current provisions.

2 MATERIALS AND METHODS

VecTor2 (Wong *et al.*, 2013) is a state-of-the-art nonlinear finite element modelling program for plane RC sections that is based on the disturbed stress field model (Vecchio *et al.*, 2000). VecTor2 has been used in a variety of past research for modelling RC walls (Bohl & Adebar, 2011; Dai, 2011; Ghorbani-Renani *et al.*, 2009; Hoult *et al.*, 2017; Lu *et al.*, 2014; Luu *et al.*, 2013; Sritharan *et al.*, 2014). In order to validate the use of VecTor2, some wall models have been developed and compared with some experimental data. To validate the material models, it is important to show that VecTor2 can predict, to a high degree of accuracy, the force-displacement response of the wall as well as the cracking and strain distribution up the height of the wall. Testing has been very limited on lightly reinforced and unconfined RC walls. However, the RC wall specimens 'Wall1' and 'Wall2' are lightly reinforced concrete walls that were tested by Albidah (2016) and Altheeb (2016) using D500N bars, which are suitable for the purposes of validating the chosen material models that will be used in VecTor2. The constitutive and material models that will be used in VecTor2 are given in Table 1. For sake of brevity, these material models are not discussed in this paper, and the reader is referred to Hoult *et al.* (2017) and Wong *et al.* (2013) for more information. After validating VecTor2, a RC wall with different longitudinal reinforcement layouts is introduced and modelled in VecTor2. The results of the VecTor2 analyses are then presented.

Table 1 Constitutive models to be used in the VecTor2 analyses

Constitutive Behavior	Model
Compression Pre-Peak	Popovics NSC[1]/Popovics HSC[2]
Compression Post-Peak	Popovics NSC[1]/Popovics HSC[2]
Compression Softening	Vecchio 1992-B (e1/e0-Form)
Tension Stiffening	Modified Bentz 2003
Tension Softening	Bilinear
FRC Tension	Not Considered
Confined Strength	Kupfer/Richter
Dilation	Variable - Kupfer
Cracking Criterion	CEB-FIP
Crack Stress Calc	Basic (DSFM/MCFT)
Crack Width Check	Agg/2.5 Max Crack Width (Deafult)
Crack Slip Calc	Walraven (Monotonic)
Creep and Relaxation	Not Available
Hysteretic Response	Palermo 2002 (w/ Decay)

[1]for concrete strength < 45MPa

[2]for concrete strength ≥ 45MPa

2.1 *Wall specimens 'Wall1' and 'Wall2'*

An experimental program from Albidah (2016) and Altheeb (2016) focused on the seismic performance of unconfined rectangular RC walls with longitudinal reinforcement layouts typical of low-to-moderate seismic regions, such as Australia. The walls incorporated different levels of longitudinal reinforcement, with two of the walls having cross-sections shown in Figure 1. The corresponding ρ_{wv} for Wall1 and Wall2 were 0.33% and 0.66% respectively. The effective height (H_e) of the two walls was 2.65 m (with A_r = 2.94) and the axial load ratio (*ALR*) was held constant at 5% (Albidah, 2016; Altheeb, 2016). The measured f_y and f_u of the D500N bars (d_{bl} of 10 mm) used in both walls were 500 MPa and 720 MPa respectively, with a yield strain (ε_{sy}) and ultimate strain (ε_{su}) of 0.29% and 10.0% respectively. An explanation is given in Priestley *et al.* (2007) that using the ε_{su} found from monontonic testing is inappropriate for moment-curvature analysis for cyclic behaviour (due to possible low cycle fatigue), which could be further extrapolated as being inappropriate for assessment purposes. Therefore, 0.6ε_{su} will be used as suggested by Priestley *et al.* (2007), where the final steel strain value used is 6.0%. The average concrete compressive strengths at the time of testing for specimens Wall1 and Wall2 were 35.2 MPa and 34.7 MPa respectively.

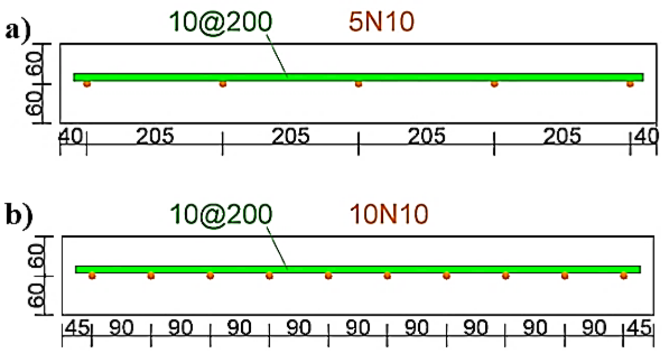

Figure 1. Cross-section and reinforcement layout of specimens (a) Wall1 and (b) Wall2 (dimensions in millimetres)

VecTor2 was used to model Wall1 and Wall2. Plane stress rectangular elements were used to model the concrete wall, which are four-noded elements with uniform thickness (Wong *et al.*, 2013). The 3:2 aspect ratio recommendation by Wong *et al.* (2013) for the plane stress elements was also adhered to. The force-displacement relationships for Wall1 and Wall2 were determined from the VecTor2 analyses using monotonic loading and are given in Figure 2 and Figure 3 respectively. Superimposed in these figures are the experimentally observed (envelope) force-displacement relationships from Altheeb (2016). Good correlations of the force-displacement relationship have been achieved from the VeTor2 models of Wall1 and Wall2 and using smeared reinforcement in comparison to that observed experimentally.

Using Equation 1, the $\rho_{wv.min}$ for both specimen Wall1 and Wall2 was found to be approximately 0.45%. It should be noted that, for the purposes of assessment, the $f'_{ct.fl}$ is approximated using the expressions given in fib (2012). This value of $\rho_{wv.min}$ indicates that specimen Wall1 had an insufficient amount of longitudinal reinforcement to onset secondary cracking, while Wall2 had a sufficient amount. This conforms with the experimental observations, where the seismic performance of the Wall1 was governed by strain penetration, with a large, primary crack forming at the base of the wall. The VecTor2 cracking distribution results in Figure 4(a) agree with the experimental observations, where the strains are primarily concentrated at the crack at the base of Wall1. This in contrast to specimen Wall2, with a higher reinforcement content, which experienced more flexural cracking in comparison to Wall1 (Figure 4b); flexural deformations primarily contributed to the overall displacement capacity of specimen Wall2 (Altheeb, 2016).

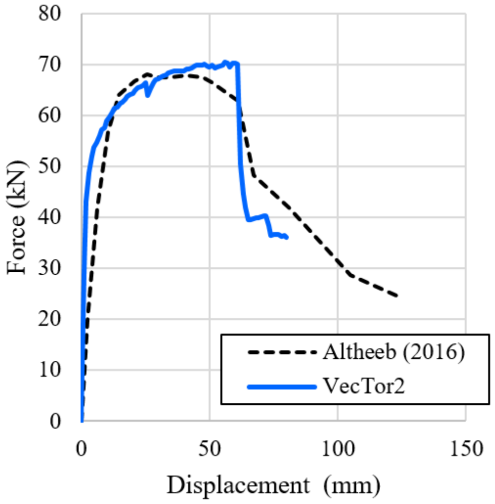

Figure 2. Force-displacement results from VecTor2 for Wall1 from Altheeb (2016)

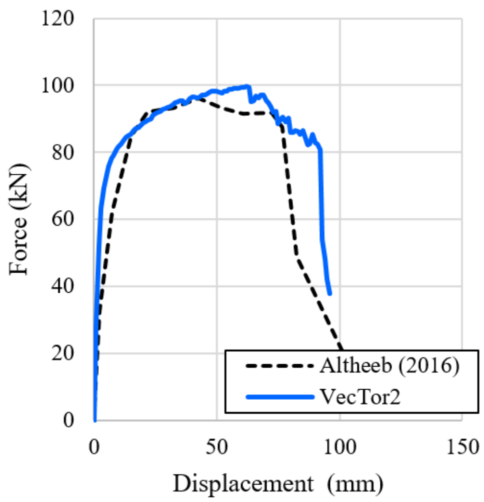

Figure 3. Force-displacement results from VecTor2 for Wall2 from Altheeb (2016)

Overall, VecTor2 has been shown to provide well correlated results to the experimental observations from the lightly reinforced wall specimens Wall1 and Wall2 (Albidah, 2016; Altheeb, 2016). Furthermore, it was shown in Hoult *et al.* (2017) that VecTor2 could produce good estimates of the force-displacement relationship and strain distribution in comparison to the experimental results of lightly reinforced concrete specimens from Oesterle *et al.* (1976) and Lu *et al.* (2015). Therefore, numerical analyses have been conducted on a lightly reinforced concrete wall with different longitudinal reinforcement layouts using the same material models in VecTor2.

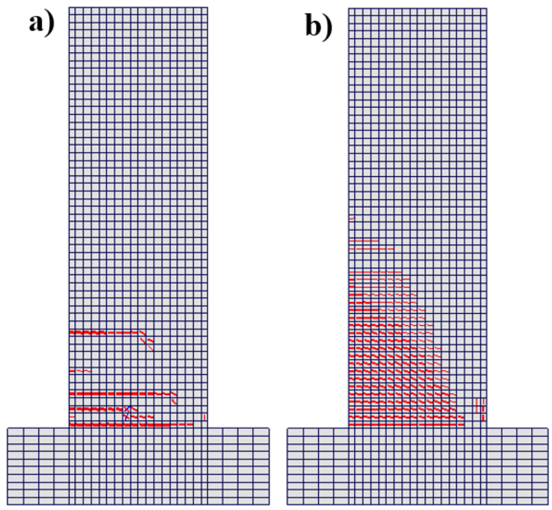

Figure 4. Cracking distributions at ultimate displacements for (a) Wall1 and (b) Wall2 from Altheeb (2016)

2.2 *Walls with different reinforcement layouts*

An RC wall was analysed in VecTor2 using different amounts of longitudinal reinforcement, both distributed and "lumped". The RC wall length (L_w) is 4 metres long, 200 mm thick and has an effective height of 9.8 metres (corresponding to a 4-storey building with a height at roof level of 14.0 metres, assuming a 3.5 metre inter-storey height). The corresponding aspect ratio ($A_r = H_e/L_w$) was thus 2.45, meaning that the wall should be primarily governed by flexural behaviour. The mean in situ compressive strength of the concrete (f_{cmi}) was assumed to be 50 MPa. This is considered a reasonable strength given that RC walls are typically design for 32 MPa or 40 MPa, where the assumption is that the strength has increased due to the ratio of actual-to-targeted concrete strength and also aging of the concrete. This would result in an increased strength factor ($\kappa = f_{cmi} / f'_c$) of approximately 1.25 to 1.6, which is consistent with the research presented by Moehle (2015) of concrete strength gained with age. Furthermore, the concrete strengths of some structural elements from the Pyne Gould and Gallery Apartment buildings in Christchurch were much higher than the design f'_c, with κ values as high as 2.4 (Hyland, 2011; Smith & England, 2012). The mean mechanical values of D500N reinforcing steel have been adopted from the tests conducted by Menegon *et al.* (2015), which are summarised in Table 2. It should be noted that $0.6\varepsilon_{su}$ was used for the ultimate strain of steel in VecTor2 due to the reasons given previously from Priestley *et al.* (2007). An assumed transverse reinforcement ratio (ρ_{wh}) value of 0.25% was used for all walls, which corresponds to the minimum required by AS 3600:2009 (Standards Australia, 2009). Both the transverse and longitudinal reinforcement were modelled using different amounts of smeared reinforcement in the concrete plane stress rectangular elements. A foundation size of 800 mm × 800 mm × 5600 mm is used for the VecTor2 models. In order to capture the expected highly localised strains in a single crack, a minimum element size corresponding to 0.5t was adopted for the walls. A refined mesh size (100 mm × 100 mm) was used for the lower section of the walls (over a height of $0.5H_e$), whereas the rectangular elements above this zone had a vertical mesh size increase (100 mm × 245 mm) to reduce the number of nodes and elements required. This decreased the computation time while improving the accuracy of cracking distributions (vertically and horizontally) at the base of the wall. This approach has been used successfully by other researchers (Bohl & Adebar, 2011; Hoult *et al.*, 2017). The 3:2 aspect ratio recommendation by Wong *et al.* (2013) for the plane stress elements was also adhered to for the bottom half of the wall. An axial load of 2000kN was applied to the walls by distribution of the force to all of the nodes at the top of the wall; this corresponds to an *ALR* of 5%. The lateral loading was also applied to all of the nodes at the top of the wall, where a 1 mm displacement was monotonically increased until failure of the wall.

Table 2 Mean values of D500N reinforcement from Menegon *et al.* (2015)

f_y (MPa)	f_u (MPa)	ε_{sh}	ε_{su}	$0.6\varepsilon_{su}$
551	660.5	2%	9.5%	6%

The six different longitudinal reinforcement configurations that will be analysed in VecTor2 are summarised in Table 3 and illustrated in Figure 5. The first three walls (W1, W2 and W3) have reinforcement detailing that is evenly distributed, corresponded to the current design practice in low-to-moderate seismic regions such as Australia (Hoult *et al.*, 2017). The first longitudinal reinforcement configuration for the RC wall ("W1") has a ρ_{wv} corresponding to the minimum required by the current AS 3600:2009 (Standards Australia, 2009), which is 0.18%. This value is based on a single-grid of 12 mm diameter bars evenly distributed and spaced at approximately 330 mm (1 × 13 layers of 12 mm bars), which also conforms to the requirements in AS 3600:2009 (Standards Australia, 2009). It should be noted that a cover distance (d_c) of 40 mm has been assumed for all the walls. W2 has the same number of layers and spacing of bars as W1, but with two grids (2 × 13 layers of 12 mm bars), corresponding to a ρ_{wv} of 0.37%. Using Equation 2, the minimum ρ_{wv} to allow secondary cracking for this wall was estimated to be 0.80% (using a f'_c of 32 MPa and f_y of 500 MPa). Therefore, W3 had a distributed longitudinal reinforcement layout (two grids) corresponding to a ρ_{wv} of 0.95% (2 × 33 layers of 12 mm bars). The other three walls (W4, W5 and W6) were detailed with "lumped" (or concentrated) longitudinal reinforcement at the boundary ends, which has been common practice in some regions of high seismicity. W4 had reinforcement in the boundary ends corresponding to a longitudinal reinforcement ratio in the boundary region (denoted in Table 3 as $\rho_{wv.b}$) of 0.94% (2 × 5 layers of 12 mm bars). Furthermore, a longitudinal reinforcement ratio in the web of the wall (denoted in Table 3 as $\rho_{wv.w}$) of 0.18% was used (1 × 9 layers of 12 mm bars), which is the minimum according to the requirements of current AS 3600:2009 (Standards Australia, 2009). W5 was detailed using the proposed longitudinal reinforcement requirements for the next revision of AS 3600. A $\rho_{wv.b}$ of 0.94% was used in the boundary ends (2 × 5 layers of 12 mm bars) and a $\rho_{wv.w}$ of 0.40% was used for the web (2 × 10 layers of 12 mm bars). Finally, W6 had the same longitudinal reinforcement detailing as W5; however, confinement was provided in the boundary regions of the wall such that the ρ_{wh} was estimated to be approximately 1.00% in both transverse directions (including "out-of-plane"). The detailing required to achieve this level of the transverse reinforcement would correspond to each layer of the longitudinal bars in the boundary region of the wall being restrained using internal fitments (with 135° hooks) and also an external fitment. It should be noted that the transverse reinforcement pictured in Figure 5 for W6 has only been provided for illustrative purposes.

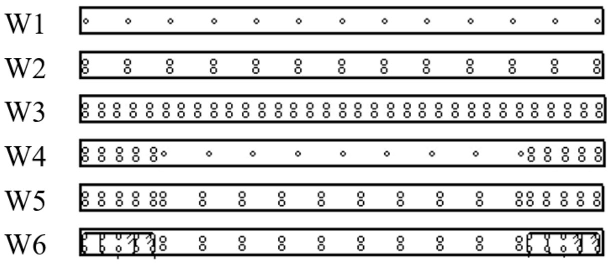

Figure 5. Longitudinal reinforcement layouts of W1 to W6

Table 3 Different reinforcement ratios of W1 to W6

	ρ_{wv}	$\rho_{wv.b}$	$\rho_{wv.w}$	No. of bars
W1	0.18%	-	-	13
W2	0.37%	-	-	26
W3	0.95%	-	-	66
W4	-	0.94%	0.18%	29
W5	-	0.94%	0.40%	40
W6	-	0.94%	0.40%	40

3 RESULTS

Figure 6 gives the force-displacement results for the 4 metre walls analysed in VecTor2 that have distributed longitudinal reinforcement (W1, W2 and W3). The circle markers in Figure 6 indicates "failure" of the walls, which correspond to the ultimate strains being reached or exceeded. The ultimate strains in the steel or unconfined concrete for this research correspond to 0.05 and -0.003 respectively. More information on the chosen strain values, which represent the "Collapse Prevention" performance level for unconfined RC walls, can be found in Hoult *et al*. (2018), Hoult *et al*. (2017) and Hoult *et al*. (2015). Figure 7 gives the cracking distribution results from VecTor2 for all of the walls analysed in VecTor2 with distributed longitudinal reinforcement (W1, W2 and W3).

Walls W1 and W2, with distributed longitudinal reinforcement, were found to be governed by tension strains, reaching a strain of 0.05 in the extreme tension fibre region of the wall at ultimate displacement capacities (Δ_u) of 19 mm and 24 mm respectively. These displacements correspond to average drifts ($\delta = \Delta_u/H_e$) of just 0.19% and 0.24% respectively. Both of these walls also had a concentration of strains at the base of the wall, where W1 formed one primary crack and W2 formed two primary cracks, shown in Figure 7(a) and Figure 7(b). This cracking distribution is consistent with that estimated using Equation 2, suggesting that these walls had an insufficient amount of longitudinal reinforcement to allow secondary cracking. Wall W3 also had distributed reinforcement but with a higher ρ_{wv} (of 0.95%) in comparison to W1 and W2. This value of ρ_{wv} for W3 is also higher than the required $\rho_{wv.min}$ (of approximately 0.80% using Equation 2), which is why secondary cracking can be observed at Δ_u in Figure 7(c) for this wall. However, the displacement capacity was still limited, as the unconfined concrete strain (of -0.003) controlled the failure of W3 at a Δ_u of 41 mm (δ of 0.43%).

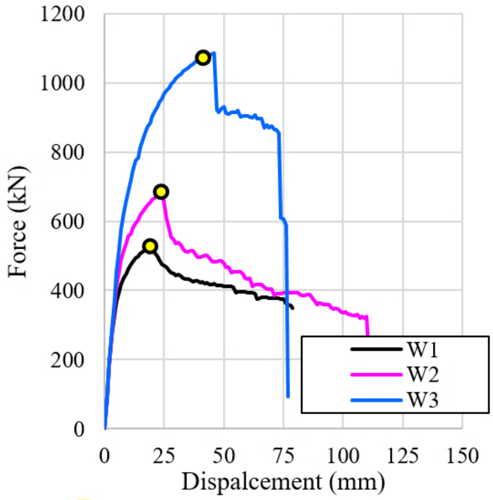

Figure 6. Force-displacement results from VecTor2 for walls with distributed longitudinal reinforcement

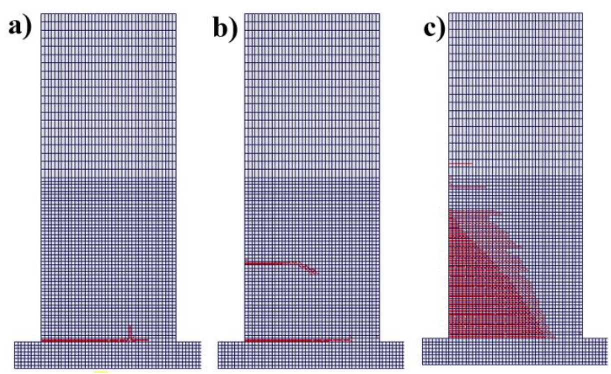

Figure 7. Cracking distribution results from VecTor2 for walls (a) W1 (b) W2 and (c) W3

Figure 8 gives the force-displacement results for the 4 metre walls analysed in VecTor2 that have lumped longitudinal reinforcement (W4, W5 and W6), whereas Figure 9 gives the cracking distribution results for these same walls. It should be noted that as some confinement was provided for W6, which was carried out by increasing the smeared transverse reinforcement in the boundary regions of the wall in VecTor2, a confined concrete strain value of -0.01 was used to indicate "failure" of this wall. This value of the concrete strain was conservative in comparison to that estimated using the expression and common values given in Paulay and Priestley (1992).

Although some secondary cracking was observed for wall W4, which had lumped longitudinal reinforcement at the boundary ends, the displacement capacity was limited due to the unconfined concrete strain being reached in the extreme compression fibre region of the wall. It is also interesting to note that a concentration of tensile strains was observed to occur for wall W4 between the junction of the boundary end (in tension) and the web of the wall. This concentration of strain will be discussed further in the next section. Wall W5 was observed to form secondary cracks and a good distribution of strains up the wall height. Wall W5 failed with the unconfined compression strain being reached at a Δ_u of 43 mm (δ of 0.44%). While the Δ_u of W5 is similar to that of W3, it should be noted that W5 had less longitudinal reinforcing bars (40 in comparison to 66, respectively, as indicated in Table 3). Wall W6 had the largest displacement capacity of the walls, which was estimated by VecTor2 to be governed by an ultimate (confined) compression strain of -0.010 at the Δ_u of 127 mm (δ of 1.3%).

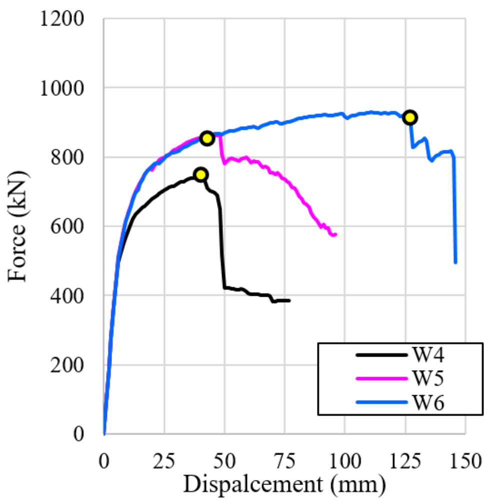

Figure 8. Force-displacement results from VecTor2 for walls with lumped longitudinal reinforcement

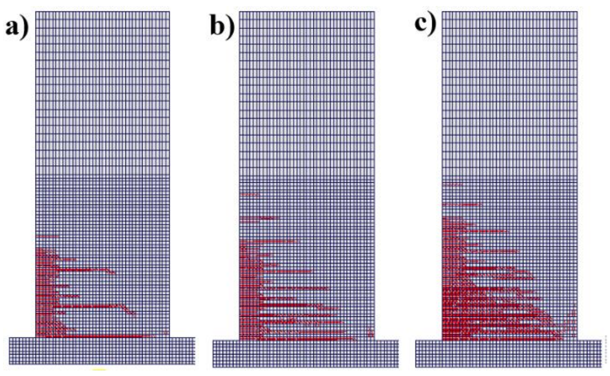

Figure 9. Cracking distribution results from VecTor2 for walls (a) W4 (b) W5 and (c) W6

A summary of the ultimate displacement capacities (and average drifts) for all walls analysed in VecTor2 are given in Table 4.

4 DISCUSSION

An interesting observation from the VecTor2 results of the RC wall with lumped longitudinal reinforcement was the concentration of tensile strains in the junction between the boundary ends of the wall and the web. This concentration of strain was observably more severe for the RC walls that had a longitudinal reinforcement ratio being considerably reduced in the web ($\rho_{wv.w}$) relative to the boundary ends ($\rho_{wv.b}$). For example, the tensile strains in wall W4, which had $\rho_{wv.b}$ and $\rho_{wv.w}$ of 0.94% and 0.18% respectively, almost reached the ultimate strain in the steel (0.05) when the unconfined concrete strain was reached in the extreme compression fibre region of the wall. The distribution of longitudinal strains at the base of wall W4 at the Δ_u is illustrated in Figure 10. In contrast, the tensile strains in wall W5, which had $\rho_{wv.b}$ and $\rho_{wv.w}$ of 0.94% and 0.40% respectively, were almost half of that of W4 at a similar level of ultimate displacement capacity. The strain distributions at the base of wall W5 at the Δ_u is illustrated in Figure 11. Poor wall performance at the junction of the wall boundary and web has also been observed in previous earthquake events. For example, walls with a light amount of longitudinal reinforcement in the web of the wall and higher concentrations of reinforcement at the boundary ends were observed to perform poorly in the Christchurch earthquake of 2011, with crushing of the wall web region (Rosso *et al.*, 2014). This damage pattern was also observed experimentally in wall tests that included a number of wall specimens with lumped longitudinal reinforcement layouts (Brueggen, 2009). Numerical analyses from Rosso *et al.* (2014) also showed that larger crack widths were expected to occur in walls with a lightly reinforced web region and with

Table 4 Summary of ultimate displacements from the walls analysed in VecTor2

	Δ_u (mm)	δ (%)
W1	19	0.19%
W2	24	0.24%
W3	41	0.42%
W4	40	0.41%
W5	43	0.44%
W6	127	1.30%

concentrated reinforcement at the boundary ends when compared with the same specimens with a distributed reinforcement layout. The larger crack widths in the web could also lead to a premature sliding failure at the base of the wall. It has been proposed that the next revision of AS 3600 increase the ρwv in the web of limited ductile RC walls to be at least half of that determined by Equation 2 for the boundary ends of the wall. While this might be sufficient for limited ductile structural walls, it is recommended that more research be conducted to investigate the seismic performance of RC walls with lumped longitudinal reinforcement in the boundary regions. In particular, a study focusing on the strain concentration phenomenon at the junction of the boundary ends and web of the wall needs to be investigated, with an emphasis on experimental testing.

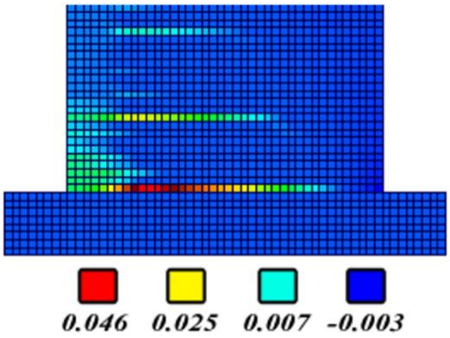

Figure 10. Concentrated longitudinal strains at ultimate displacement for W4

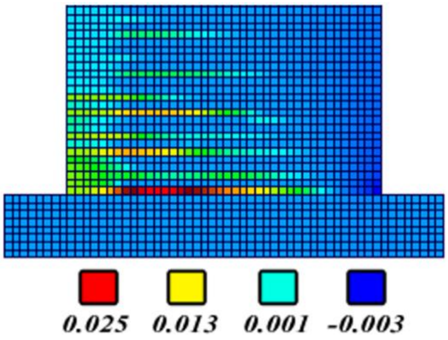

Figure 11. Concentrated longitudinal strains at ultimate displacement for W5

5 CONCLUSIONS

Poor performance of lightly reinforced concrete walls has been observed in recent earthquake events. This paper highlights the importance of providing a sufficient amount of longitudinal reinforcement to allow secondary cracking. The finite element modelling results of walls W1 and W2 showed that providing low amounts of evenly distributed longitudinal reinforcement ratios in RC walls, such as the minimum requirements in AS 3600:2009, can lead to a concentration of the longitudinal strains that prohibit the displacement (and drift) capacity. A better performance and displacement capacity was reached for the RC wall with a sufficient amount of evenly distributed longitudinal reinforcement to allow secondary cracking (wall W3), determined by Equation 1. However, the results from the RC walls analysed here with “lumped” longitudinal reinforcement (walls W4 and W5) achieved a similar ultimate displacement (or drift) capacity in comparison to wall W3 but requiring far less steel. Furthermore, it was shown that a sufficient amount of longitudinal reinforcement in the boundary region of the wall to cause secondary cracking could be achieved using Equation 2, which was derived from Equation 1 using conservative design values such that it can be implemented in the next revision of AS 3600. The results from VecTor2 for the walls detailed with boundary elements also showed a concentration of tensile strains in intersection of the web and boundary ends. Furthermore, these concentrated strains reduced by approximately a half in value by increasing the longitudinal reinforcement ratio in the web by a factor of 2 (wall W4 compared to W5). While more research is needed in this area, it is recommended that the web of RC walls with boundary regions are detailed with (at least) half of that required with Equation 2. The numerical results also showed that while some walls had sufficient longitudinal reinforcement to allow secondary cracking, the displacement capacity was still limited due to lack of confinement in the boundary regions of the wall. This was further emphasized with the ultimate displacement capacity achieved by wall W6 with confined boundary ends, the value of which was approximately three times greater than the same wall without confinement (wall W5). Thus, it is also recommended that if a RC structural wall is to be designed for some ductility (e.g. greater than 2), confinement must be provided in the boundary regions of the wall. More research is needed on different longitudinal reinforcement layouts to ensure an ideal configuration is used. In particular, three key areas could be investigated further: (i) confinement requirements for the boundary regions of limited ductile walls for AS 3600, (ii) the minimum longitudinal reinforcement ratio in the web compared to the boundary ends and (iii) reinforcement layouts using different mechanical properties of the reinforcing steel (e.g. D500E or high-strength steel).

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