MINIMUM VERTICAL REINFORCEMENT IN RC WALLS: THEORETICAL REQUIREMENTS FOR LOW AND HIGH DUCTILITY DEMANDS

Yiqiu Lu[1] and Richard S. Henry[2]

ABSTRACT

Recent earthquakes and research have shown that the minimum vertical reinforcement requirements in current concrete standards are insufficient to ensure well distributed cracking occurs in ductile reinforced concrete (RC) walls. To address the deficiencies of existing requirements, new theory was proposed to calculate the minimum distributed and end zone vertical reinforcement required for RC walls to meet current performance expectations. The distributed vertical reinforcement requirement was intended to prevent non-ductile behaviour for walls with low ductility demands, and was derived based on the requirement that nominal flexural strength must exceed the cracking moment capacity. The vertical reinforcement required in the ends of the wall was intended to ensure that well distributed secondary cracks form in the plastic hinge region of walls with high ductility demands, and was derived to ensure that the concrete tensile strength could be overcome by the tensile demands imposed when the vertical reinforcement in the ends of the wall yields. The proposed requirements considered the key parameters that influence the behaviour of walls with minimum vertical reinforcement. In addition, the proposed formulas were compared with current minimum vertical reinforcement limits from different concrete design standards by considering the margin of safety between cracking and nominal flexural strength and the secondary cracking behaviour. The deficiencies of the existing requirements were demonstrated and the proposed requirements were proved to be superior for walls with both low and high ductility demands.

INTRODUCTION

Minimum vertical reinforcement requirements for reinforced concrete (RC) walls are imposed by most concrete design standards worldwide, in part to mitigate shrinkage and temperature effects, but also to prevent non-ductile failure modes [1]. However, observations from recent earthquakes and research have demonstrated that the seismic performance of RC walls designed in accordance with both former and current minimum vertical reinforcement requirements can be unsatisfactory. For example, during the 2010/2011 Canterbury earthquakes in New Zealand, several RC walls in multi-storey buildings that were designed in accordance with minimum vertical reinforcement requirements from former versions of the New Zealand Concrete Structures Standard, NZS 3101, formed only a limited number of cracks in the plastic hinge region as opposed to the expected distributed cracking [2, 3]. Following the Canterbury earthquakes, a series of experimental tests and numerical models were conducted to investigate the behaviour of RC walls with minimum vertical reinforcement in accordance with current concrete design standards both in New Zealand and worldwide [4-7]. Results indicated that the minimum vertical reinforcement requirements in current concrete standards are insufficient to ensure that well distributed cracking occurs in the plastic hinge region of ductile RC walls.

To address the deficiencies identified with existing requirements, the development of a fundamental theory for minimum vertical reinforcement requirements for RC walls is considered essential. Equations were developed to represent the minimum vertical reinforcement for RC walls required to ensure that design objectives are achieved for walls with both low and high ductility demands. The proposed formulas were

verified against existing experimental data and numerical modelling results and compared with existing requirements for minimum vertical reinforcement in different concrete design standards.

PERFORMANCE OF RC WALLS DESIGNED TO CURRENT LIMITS

Table 1 lists the minimum vertical reinforcement requirements for RC walls with different ductility demands in six concrete standards including NZS 3101:2006 (Amendment 2) (A2) [8], NZS 3101:2006 (Amendment 3 draft) (A3 draft) [9], the US Building Code Requirements for Structural Concrete, ACI 318-14 [10], Eurocode 8: Design of structures for earthquake resistance [11], the Canadian Design of Concrete Structures standard, CSA A23.3-14 [12] and the Chinese Code for Design of Concrete Structures, GB 50010-2010 [13]. A series of experimental tests and numerical analyses were conducted to evaluate these minimum vertical reinforcement requirements [4-6]. Key results from the tests and analyses are summarised below to compare the seismic behaviour of walls designed in accordance with different design standards.

New Zealand Concrete Standard

Prior to the 2010/2011 Canterbury earthquakes, NZS 3101:2006 (A2) only required a minimum total vertical reinforcement ratio of $\sqrt{f'_c}/4f_y$, with no requirement for additional vertical reinforcement to be placed at the ends of the wall [8]. This vertical reinforcement ratio was required for all RC walls irrespective of the seismic ductility demands, as shown in Table 1. Following the observations in the 2010/2011 Canterbury earthquakes, new amendments were proposed to the

[1] *Research Fellow, Dept. of Civil and Environmental Engineering, University of Auckland, New Zealand: ylu765@aucklanduni.ac.nz*
[2] *Senior Lecturer, Dept. of Civil and Environmental Engineering, University of Auckland, New Zealand: rs.henry@auckland.ac.nz (Member)*

minimum vertical reinforcement requirements in NZS 3101:2006 (A3 draft) [9] for ductile or limited ductile hinge regions. The proposed amendments require additional vertical reinforcement to be placed at the ends of the wall (end zone length defined as $0.15l_w$) with a reinforcement ratio of at least $\sqrt{f'_c}/2f_y$.

Table 1: Minimum vertical reinforcement requirement for RC walls with low and high ductility demands.

Design Standard	Low ductility demands[1]		High ductility demands[2]	
	Total/distributed reinforcement ratio	End zone reinforcement ratio	Total/distributed reinforcement ratio	End zone reinforcement ratio
NZS 3101:2006 (A2)	$>\sqrt{f'_c}/(4f_y)^*$	No requirement	$>\sqrt{f'_c}/(4f_y)^*$	No requirement
NZS 3101:2006 (A3 draft)	$>\sqrt{f'_c}/(4f_y)^*$	No requirement	$>\sqrt{f'_c}/(4f_y)^*$	$>\sqrt{f'_c}/(2f_y)^*$
ACI 318-14	>0.15%	No requirement	>0.25% (or >0.15%)	No requirement
Eurocode 8	>0.2%	>0.5%	>0.2%	>0.5%
GB 50010-2010	>0.2%	>0.5%	>0.25%	>1.0 %
CSA A23.3-14	>0.25%	No requirement	>0.25%	$>(0.15\% t_w l_w)/(t_w l_e)$

[1] Low ductility defined as nominally ductile (μ=1.25) in NZS 3101, ordinary structural walls in ACI 318, medium ductility class in Eurocode 8, seismic level III and IV in GB 50010, and moderately ductile walls in CSA A23.3

[2] High ductility defined as limited ductile or ductile plastic regions in NZS 3101, special structural walls in ACI 318, high ductility class in Eurocode 8, seismic level I and II in GB 50010, and ductile plastic hinge regions in CSA A23.3

* f'_c and f_y in MPa

Six RC walls designed in accordance with the minimum vertical reinforcement requirements in NZS 3101:2006 (A2) were tested by Lu et al. [4]. The typical behaviour of an example test wall C2 (shear span ratio $M/Vl_w = 4$ and axial load ratio = 3.5%) is shown in Figure 1. The test results showed that despite exhibiting some ductility, the behaviour of all six test walls was controlled by 1-3 large flexural cracks at the wall base. The distribution of curvature and reinforcement strains in the plastic hinge region of the test walls was not linear, but instead concentrated at the locations of wide flexural cracks. The limited cracking greatly reduced the spread of the plasticity and ductility. In addition, the results of finite element analyses showed that the cracking behaviour and drift capacity of RC walls with the minimum distributed vertical reinforcement in accordance with NZS 3101:2006 (A2) was strongly influenced by wall size, reinforcement properties, and concrete strength [5]. The experimental and modelling results both showed that the minimum distributed vertical reinforcement requirements in NZS 3101:2006 (A2) are only suitable for walls designed for low ductility demands.

A second series of laboratory tests were conducted on five RC walls designed with minimum vertical reinforcement in accordance with the new requirements proposed in NZS 3101:2006 (A3 draft) [6]. The results of an example test wall M1 (shear span ratio $M/Vl_w = 4$ and axial load ratio = 3.5%) is shown in Figure 2. In contrast to the comparable wall C2 that had minimum distributed reinforcement as per NZS 3101:2006 (A2), the cracks in wall M1 were more evenly distributed over the plastic hinge region. The curvatures and reinforcement strains were also more evenly distributed over the wall height. The test results showed that the additional vertical reinforcement limit proposed in the end zone of ductile walls in NZS 3101:2006 (A3 draft) was sufficient to ensure that well distributed secondary cracks occurred in the plastic hinge region and suitable for limited ductile and ductile plastic hinge regions.

Concrete Standards Worldwide

In addition to New Zealand Concrete Standards, the minimum vertical reinforcement requirements for RC walls in other concrete standards (as listed in Table 1) were also reviewed and examined by Lu [6]. The provisions for minimum vertical reinforcement differ substantially with regards to reinforcement ratio limits and distribution requirements. A comprehensive numerical study was conducted on the behaviour of walls with minimum vertical reinforcement in accordance with requirements in each design standard for walls with high ductility demands, and the results are shown for comparable walls in Figure 3 (shear span ratio $M/Vl_w = 3$, axial load ratio = 3.5%, concrete strength $f'_c = 38.5$ MPa, reinforcement yield strength $f_y = 554$ MPa).

The RC walls modelled with a fixed distributed minimum reinforcement ratio of 0.25% or less (as per ACI 318-14) could not generate a large number of distributed cracks in the wall plastic hinge, resulting in premature reinforcement fracture and low drift capacities. Concentrating a greater portion of the reinforcement at the ends of the wall improved the cracking behaviour and ductility. However, RC walls with end zone and distributed reinforcement ratios of 0.5% and 0.2% respectively (as per Eurocode 8) did not exhibit sufficient ductility with vertical reinforcement fracture occurring at a modest lateral drift for the modelled walls. The response of RC walls modelled with an end zone and distributed reinforcement ratio of 1.0% and 0.25% respectively (as per GB 50010-2010 and CSA A23.3-14) were significantly better than the walls with an end zone and distributed reinforcement ratio of 0.5% and 0.2% respectively. However, due to the large difference between end zone and distributed reinforcement ratios, the web region was vulnerable to the formation of wide discrete cracks, causing premature of web reinforcement fracture and large shear deformations [6]. Furthermore, higher concrete strengths, lower reinforcement strengths and lower axial loads were all found to significantly reduce the secondary cracks and deformation capacity for RC walls designed in accordance with fixed minimum vertical reinforcement limits (as per all the standards except for NZS 3101:2006).

Summary

The provisions for minimum vertical reinforcement in each standard differ substantially and the seismic behaviour also varied significantly between RC walls that conformed to each standard. Most of the minimum vertical reinforcement requirements for ductile RC walls in these concrete standards are insufficient to ensure ductile behaviour. One of the reasons for the inconsistent and unsatisfactory performance is that most of these minimum requirements were based on engineering judgement and lacked an underpinning theoretical basis. Most of the current minimum vertical reinforcement limits are fixed

quantities that are independent of axial load or material strengths. Although NZS 3101:2006 accounts for the concrete and reinforcing steel strengths and the new minimum vertical reinforcement limits proposed for ductile walls in NZS 3101:2006 (A3 draft) performed the best out of all the design standards, the NZS 3101:2006 requirements do not consider axial load, which was shown as an important factor that influenced the behaviour of nominally ductile lightly reinforced concrete walls [4, 7]. Proposing a fundamental theory for minimum vertical reinforcement limits for RC walls is considered essential to improve the seismic behaviour of lightly reinforced concrete walls.

(a) Overall condition at end of test   *(b) Final crack pattern and crack widths (mm) at 2.5% drift*   *(c) Curvature profile*
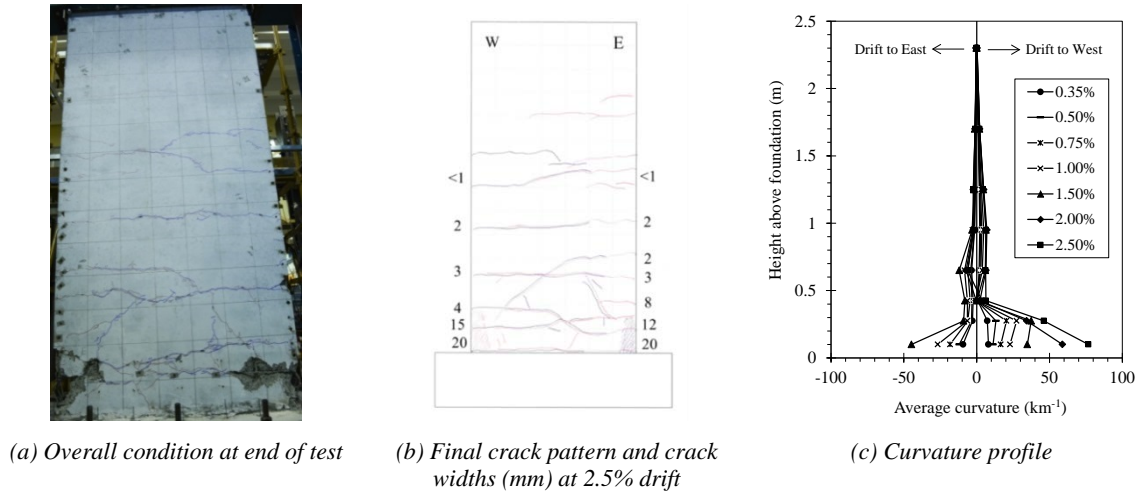
Figure 1: Experimental results of wall C2.

(a) Overall condition at end of test   *(b) Final crack pattern and crack widths (mm) at 2.5% drift*   *(c) Curvature profile*
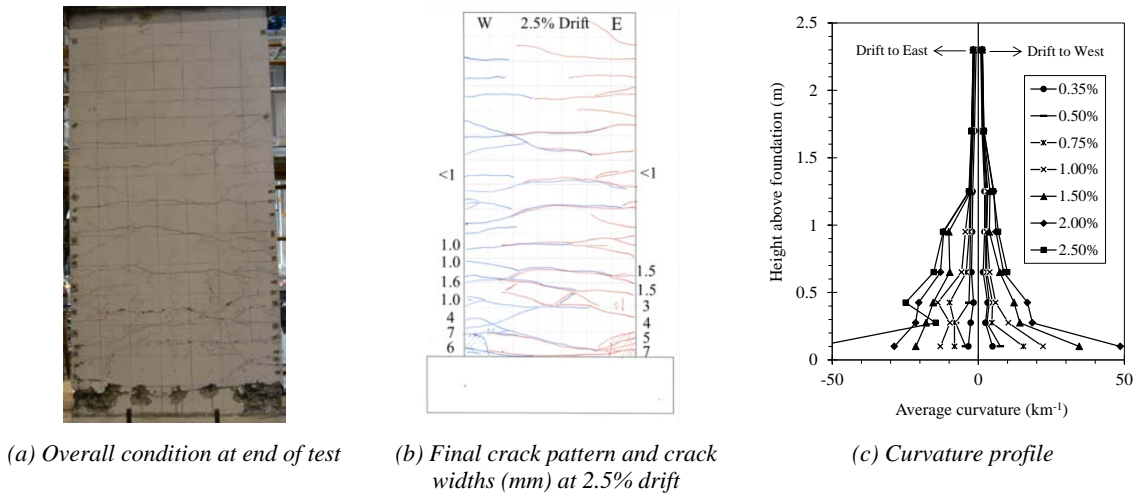
Figure 2: Experimental results of wall M1.

(a) ACI 318-14   *(b) NZS 3101: 2006 (A2)*   *(c) Eurocode 8*   *(d) CSA A23.3-14/GB 50010-2010*   *(e) NZS 3101: 2006 (A3 draft)*
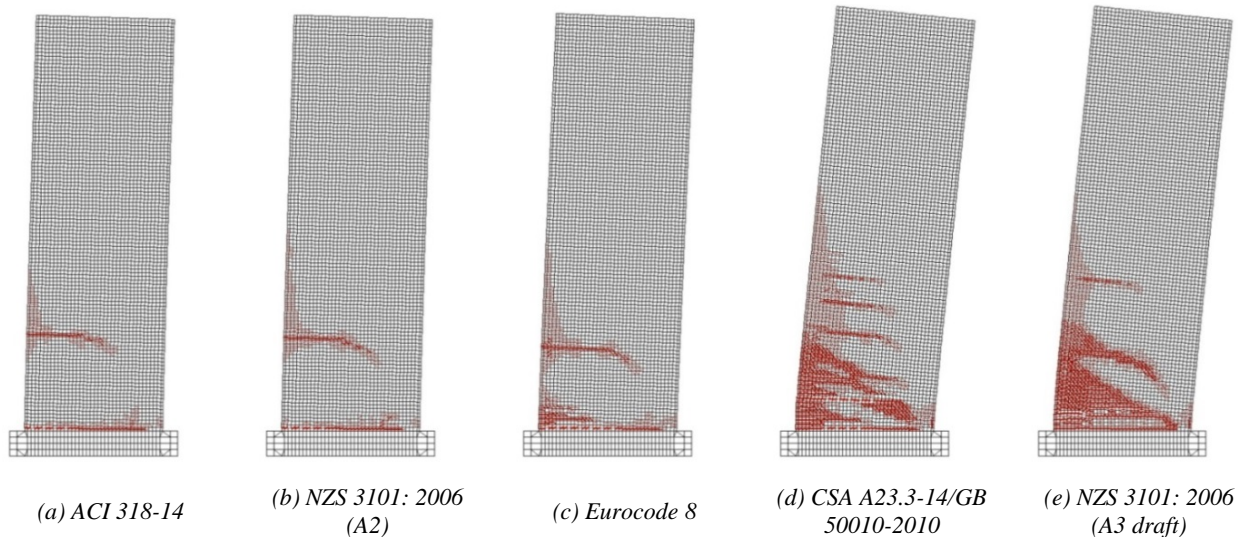
Figure 3: Typical crack patterns for modelled walls with minimum vertical reinforcement in accordance with different standards.

PROPOSED MINIMUM VERTICAL REINFORCEMENT LIMITS

In RC beams, the longitudinal reinforcement is usually located at the top and bottom of the beam section and the minimum longitudinal reinforcement refers to the tension reinforcement. However, RC walls typically have distributed vertical reinforcement through the entire wall section and in some cases reinforcement concentrated at the ends of the wall. The required minimum vertical reinforcement in RC walls can therefore be classified based on minimum distributed vertical reinforcement and minimum end zone vertical reinforcement.

For a wall with an extremely low quantity of vertical reinforcement, the cracking moment may exceed the nominal flexural strength and inelastic action is unlikely to extend beyond the section at which the first crack forms [7]. For example, experimental results showed that walls that had a cracking moment that exceeded the nominal flexural capacity formed a single crack with extremely small drift capacity [14]. In accordance with modern concrete design standards, this potential non-ductile failure mode should be prevented for all RC walls designed to resist seismic loads irrespective of design ductility. For low ductility demands, distributed cracking is not necessarily required, and the minimum vertical reinforcement can be distributed evenly along the wall length without requiring additional reinforcement concentrated in the end zone of the wall, as shown in Figure 4-a. To prevent a non-ductile response, this minimum distributed vertical reinforcement should be sufficient to ensure that the nominal flexural strength, M_n, is greater than the probable cracking moment, M_{cr} (referred to as the M_n/M_{cr} criterion).

For high ductility demands, in addition to satisfying M_n/M_{cr} criterion, RC walls must form well distributed secondary flexural cracks to ensure a good spread of plasticity in the plastic hinge region (referred to as secondary cracking criterion). The M_n/M_{cr} criterion is only suitable to prevent sudden failure due to a loss of lateral strength after cracking and does not ensure that secondary cracks develop in the plastic hinge region. RC walls that exhibit a M_n/M_{cr} ratio larger than 1 may still form discrete irregular cracks in the plastic hinge region, with concentrated plasticity at these locations that leads to premature reinforcement fracture and low ductility [4]. To ensure that the secondary cracking criterion is satisfied, additional vertical reinforcement must be placed at the ends of the wall, as shown in Figure 4-b. The tension capacity of the end zone vertical reinforcement must be larger than the tension capacity of the surrounding concrete to ensure that secondary cracks initiate.
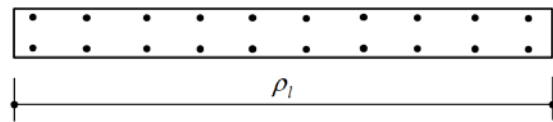
(a) Minimum distributed reinforcement for all seismic resistance walls
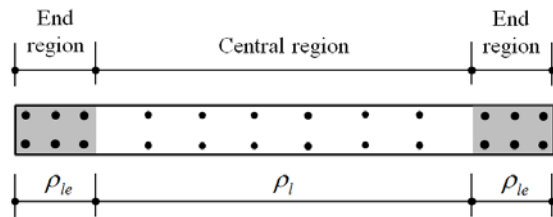
(b) Minimum end zone and distributed reinforcement for ductile walls

Figure 4: Illustration of minimum vertical reinforcement for RC walls.

Based on these performance objectives, a minimum distributed vertical reinforcement limit is proposed for all RC walls designed to resist earthquake loads and an additional end zone vertical reinforcement limit is proposed for RC walls with high ductility demands. The ductility demands could be defined using different parameters such as displacement or curvature ductility and drift capacity. For example, in NZS 3101:2006, the low ductility demand is defined as a nominal ductile structure ($\mu = 1.25$) while high ductility demand is defined as ductile or limited ductile ($\mu \leq 5$ and 3, respectively) [8]. In the following sections, the proposed minimum distributed vertical reinforcement and minimum end zone vertical reinforcement limits are described for a rectangular wall section. The shear demands are typically low in lightly reinforced concrete walls, as indicated with previous tests [4], and so the proposed theory and equations are based on flexural behaviour.

Minimum Distributed Vertical Reinforcement

As previously discussed, the minimum distributed vertical reinforcement should satisfy the M_n/M_{cr} criterion, as shown by Eq. 1, where Ω is a safety factor to account for variability in the material strengths and dynamic response.

$$M_n > \Omega M_{cr} \tag{1}$$

Cracking Moment

RC walls are usually subjected to a combination of bending and axial load. As the applied load increases the tensile stress in the extreme tension fibre of the wall will reach the concrete flexural tensile strength and the wall section will crack. The corresponding bending moment is referred to as the cracking moment, M_{cr}, and can be expressed as shown in Eq. 2, where t_w is the wall width, l_w is the wall length, f_t is the mean concrete tensile strength calculated as per the *fib* model code [15], as shown in Eq. 3. It should be noted that for typical wall lengths the flexural tensile strength is approximately equal to the direct tensile strength [15].

$$M_{cr} = \frac{t_w l_w^2}{6}(f_t + f_a) \tag{2}$$

$$f_t = f_{ct} = 0.3\left(f_c'\right)^{\frac{2}{3}} \quad \text{(MPa)} \tag{3}$$

$$f_a = n f_c' \tag{4}$$

f_a is the concrete compressive stress due to axial load, as calculated by Eq. 4, where n is the axial load ratio and f_c' is the specified concrete compressive strength.

Nominal Flexural Strength

Figure 5 shows the strain and stress distribution of a rectangular wall section subjected to a combination of an axial load, *P,* and bending moment, *M*. The calculation of the nominal flexural strength (M_n) utilised the following assumptions:

- Plane sections remains plane;
- The tensile strength of concrete is neglected (section is cracked at nominal strength);
- Ultimate compression strain in extreme compression fiber is 0.003;
- Vertical reinforcement is evenly distributed/smeared over the wall length;
- All the vertical reinforcement yields in either tension or compression.

Taking the bending moment about the point of the concrete compression resultant force, the nominal flexural strength M_n can be expressed as Eq. 5, where ρ_l is the distributed vertical

reinforcement ratio, f_y is the reinforcement yield strength, c is the concrete neutral axis length and a is the equivalent length of the rectangular stress block.

$$M_n = \rho_l f_y \left(l_w - c\right) t_w \frac{l_w + c - a}{2} + P\left(\frac{l_w}{2} - \frac{a}{2}\right) \tag{5}$$

In Eq. 5, the bending moment of compression reinforcement about the point of the concrete compression resultant force is neglected as the point of reinforcement compression resultant force is close to the point of concrete compression resultant force.
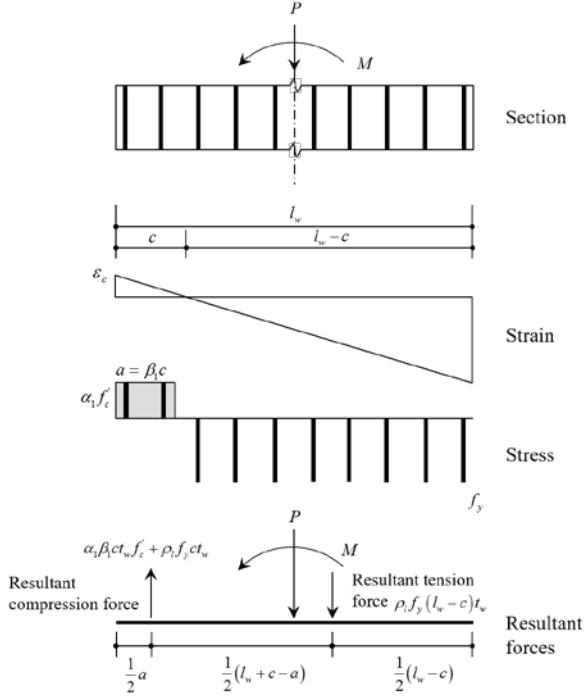
Figure 5: Rectangular RC wall section at nominal strength.

M_n/M_{cr} Criterion

By substituting Eq. 2 and Eq. 5 into Eq. 1, the minimum required distributed vertical reinforcement ratio can therefore be derived as shown in Eq. 6.

$$\rho_l \geq \frac{\Omega \frac{1}{3}\left(f_t + nf_c'\right) - nf_c'\left(1 - \frac{a}{l_w}\right)}{f_y\left(1 - \frac{c}{l_w}\right)\left(1 + \frac{c - a}{l_w}\right)} \tag{6}$$

As illustrated in Figure 5, the ratio of the wall length in compression can be expressed as defined in Eq. 7, where α_1 and β_1 are the parameters of the rectangular stress block as defined by NZS 3101:2006.

$$\frac{c}{l_w} = \frac{nf_c' + \rho_l f_y}{\alpha_1 \beta_1 f_c' + 2\rho_l f_y} \tag{7}$$

By substituting Eq. 7 into Eq. 6, the minimum distributed vertical reinforcement ratio ρ_l can be solved by a quadratic equation. However, the equation is complex that it is not easy for implementation as a design provision. As discussed by Lu [6], the most significant factor influencing c/l_w for walls with minimum distributed vertical reinforcement is the axial load ratio. A parametric study showed that the simplified expression shown in Eq. 8 can be used to estimate c/l_w with the error typically less than 10% when compared to the c/l_w calculated using Eq. 7.

$$\frac{c}{l_w} = 0.06 + n \tag{8}$$

To further simplify the equation, the equivalent length of the rectangular stress block (a) was assumed to be equal to the neutral axis depth (c), as both are typically small for lightly reinforced concrete walls and so this simplification does not significantly affect the accuracy [6]. Using the estimated neutral axis depth proposed in Eq. 8 and the assumed equivalent length of the rectangular stress block (a), the required minimum distributed vertical reinforcement ρ_l can be expressed as shown in Eq. 9.

$$\rho_l \geq \frac{\frac{\Omega}{3}\left(f_t + nf_c'\right) - nf_c'\left(0.94 - n\right)}{f_y\left(0.94 - n\right)} \tag{9}$$

As shown by Lu [6], the minimum distributed vertical reinforcement ratio calculated using Eq. 9 provided good accuracy when compared to the iterative method using Eq. 6 and Eq. 7.

Minimum End Zone Vertical Reinforcement

For RC walls designed to exhibit significant ductility, additional vertical reinforcement must be placed at the ends of the wall to ensure that well distributed secondary cracks form. Figure 6 illustrates the crack model of a RC wall with additional end zone reinforcement. The proposed approach is similar with that developed for NZS 3101:2006 (A3 draft) and reported by Cook et al. [16].
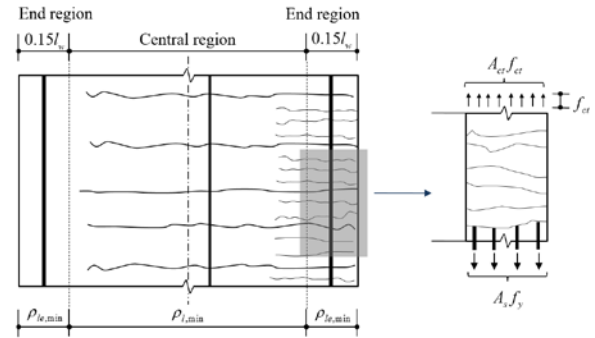

Figure 6: Cracking model for RC walls with additional end zone reinforcement.

As shown in Figure 6, the force generated when the vertical reinforcement (A_s) in the end zone reaches its yield strength (f_y) must exceed the direct tensile strength of the sounding concrete, which is equal to the area of concrete in the end zone (A_{ct}) multiplied by the concrete direct tensile strength (f_{ct}), as expressed in Eq. 10. The end zone length is usually defined as $0.15l_w$ by most of concrete standards and has been shown to be sufficient to ensure that secondary cracks that initiate propagate further along the wall length.

$$A_s f_y \geq \lambda A_{ct} f_{ct} \tag{10}$$

Assuming that the area of concrete in tension equals the gross area of the section, the end zone reinforcement ratio can be calculated as shown in Eq. 11, where f_{ct} is calculated in accordance with Eq. 3.

$$\rho_{le} \geq \lambda \frac{f_{ct}}{f_y} = \lambda \frac{0.3\left(f_c'\right)^{\frac{2}{3}}}{f_y} \tag{11}$$

A factor λ is included to accounts for the effects of dynamic loading, drying shrinkage of concrete and average long-term

material strengths. The NZS 3101:2006 (A3 draft) provisions introduced detailed factors to account for the likely material strengths in walls, including [16]:

- 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 average target compressive strength given in NZS 3104:2003 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);

The detailed factors accounting for these effects introduced in NZS 3101:2006 (A3 draft) were also adopted in the proposed requirements to replace λ, as shown in Eq. 12.

$$\rho_{le} \geq \frac{1.2 \times 0.85 \times 0.3 \left(1.2 \times 1.1 f'_c\right)^{\frac{2}{3}}}{1.1 \times 1.08 f_y} = \frac{0.3 \left(f'_c\right)^{\frac{2}{3}}}{f_y} \qquad (12)$$

Comparing Eq. 11 and Eq. 12, it is indicated that the equivalent λ when considering the combination of material modification factors was approximately equal to 1.0. If less reinforcement is provided than that suggested by ρ_{le} defined in Eq. 12, it is possible to form discrete irregular cracks in the plastic hinge region with concentrated plasticity in the wide crack locations [4]. This behaviour might result in poor performance or unexpected failure during earthquakes as the wall may not develop the assumed level of ductility used in the analysis.

TRENDS OF KEY PARAMETERS

Minimum Distributed Vertical Reinforcement

The minimum distributed vertical reinforcement ratio required by Eq. 9 accounts for three critical parameters which are concrete strength, reinforcement yield strength and axial load ratio. The trend of each of these parameters was studied and compared with the wall behaviour during experimental testing and numerical modelling in previous studies [4-6].
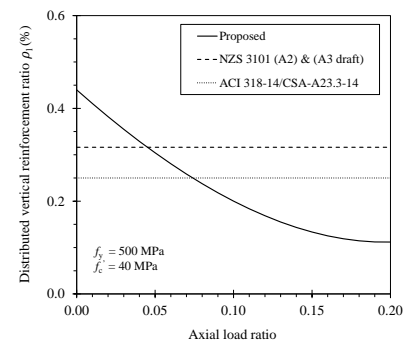
Concrete Strength

Figure 7-a shows the calculated minimum distributed vertical reinforcement ratio for concrete compressive strength ranging from 30 MPa to 70 MPa assuming reinforcement yield strength 500 MPa and axial load ratio 3.5%. The safety factor (Ω) was assumed to be 1.6, which was the average of the range of M_n/M_{cr} for minimum reinforcement applied to rectangular beams [7]. The required minimum distributed vertical reinforcement ratio increased as the concrete compressive strength increased. This trend appropriately reflected the previous observed behaviour of lightly reinforced concrete walls. Lu [6] concluded that when the vertical reinforcement content remained fixed, the secondary cracks and drift capacity of lightly reinforced concrete walls reduced significantly when using higher strength concrete. For example, the three walls that were designed in accordance with a fixed minimum vertical reinforcement ratio of 0.25%, as per ACI 318-14, were identical except for the concrete compressive strength which ranged from 38.5 MPa, 50 MPa and 60 MPa. The behaviour of the walls with concrete strengths of 38.5 MPa and 50 MPa were dominated by two primary cracks with no significant secondary cracking over the wall height, and a drift capacity of 0.57%. However, the wall with concrete strength of 60 MPa exhibited only a single crack at the wall base and the drift capacity reduced to 0.46%. Similar results can also be found in the walls that were designed in accordance with the fixed minimum vertical reinforcement requirements in Eurocode 8 and CSA A23.3-14/GB 50010-2010. The higher concrete strength results in higher cracking moment and so more reinforcement is required to improve the nominal flexural capacity, which is consistent with the theory proposed in Eq. 9. This trend was also consistent with the current minimum reinforcement limits in NZS 3101:2006, but not with the fixed distributed minimum limits employed by other concrete standards, as shown in Figure 7-a.

Reinforcing Steel Strength

Figure 7-b shows the calculated minimum distributed vertical reinforcement ratio for reinforcement yield strengths ranging from 300 MPa to 600 MPa assuming safety factor 1.6, concrete compressive strength 40 MPa and axial load ratio 3.5%. In contrast to the trend observed for concrete compressive strength, the required minimum distributed vertical reinforcement ratio decreased as the reinforcement yield strength increased. This trend also accurately reflected the behaviour of lightly reinforced concrete walls and was verified by previous numerical modelling results [6].

(a) Concrete compressive strength   *(b) Reinforcement yield strength*   *(c) Axial load ratio*

Figure 7: Minimum distributed vertical reinforcement for walls with low ductility demands

As concluded by Lu [6], when the vertical reinforcement content remained fixed, the ductility and drift capacity of lightly reinforced concrete walls were reduced when using lower yield strength reinforcement. For instance, three walls were modelled

which all had a distributed reinforcement ratio of 0.2% and an end zone reinforcement ratio of 0.5% but had different grades of reinforcement, including G500E (f_y=544 MPa), Class B (f_y=484.9 MPa) and Class C (f_y=601 MPa) reinforcement.

Compared to wall with G500E reinforcement, the cracking of wall with Class B reinforcement was reduced due to the lower yield strength of Class B reinforcement. However, the higher reinforcement yield strength of the Class C reinforcement resulted in a significant greater number of secondary cracks and reinforcement strains that were more evenly distributed in the plastic hinge region. The drift capacity of the three walls was calculated to be 0.85%, 0.32% and 0.96%, respectively. These findings confirmed that the required minimum distributed vertical reinforcement should be increased when using lower yield strength reinforcement, which is again consistent with the theory proposed in Eq. 9. As with concrete strength, the trend was also consistent with the current minimum reinforcement limits in NZS 3101:2006, but not with the fixed distributed minimum limits employed by other standards, as shown in Figure 7-b.

Axial Load Ratio

Figure 7-c shows the calculated minimum distributed vertical reinforcement ratio with axial load ratio ranging from 0 to 20% assuming the safety factor 1.6, concrete compressive strength 40 MPa and reinforcement yield strength 500 MPa. The minimum distributed vertical reinforcement ratio decreased as the axial load ratio increased. This trend is consistent with the results of moment-curvature analysis conducted by Henry [7], where it was concluded that the margin of safety between the cracking moment and ultimate flexural strength increased when higher axial loads applied to lightly reinforced concrete walls. Extremely low axial loads could cause non-ductile behaviour with cracking moment exceeding nominal flexural capacity for lightly reinforced concrete walls that could have some ductility if it was subjected to a reasonable axial load [6]. In addition, the experimental results presented by Lu et al. [4] also showed that the behaviour of test walls with an axial load ratio of 3.5% or 6.6% was controlled by 2-3 primary cracks while wall C4 with no axial load was almost entirely controlled by a single crack at the wall base. The drift capacity of wall C4 was only 1.5%, which was significantly lower than 2.5% of the test walls with axial load of 3.5% or 6.6%. From the results presented above, it can be concluded that the axial load ratio should be employed in the equation of minimum distributed vertical reinforcement, as proposed in Eq. 9. However, it should be noted that none of the minimum reinforcement limits in current standards account for axial load, as shown in Figure 7-c. The inclusion of axial load, as proposed in Eq. 9, will improve the seismic performance of RC wall with low axial loads and reduce the conservatism with minimum vertical reinforcement for walls with higher axial loads.

Minimum End Zone Reinforcement Ratio

The derived equation for end zone reinforcement (Eq. 12) is intended to ensure that secondary cracks form at the ends of wall to provide ductility during earthquakes. To investigate whether this equation can accurately reflect the trend of the cracking behaviour for RC walls, a "secondary cracking index" defined as $f_y \rho_{le}/f_{ct}$ was introduced, which is essentially just a rearrangement of Eq. 12 and gives a ratio of the vertical reinforcement strength to the concrete tensile strength in the wall end zone. As the index increases, the greater the tensile capacity of the reinforcement ($A_s f_y$) is relative to the concrete and the higher the probability of secondary cracks forming. To verify this theory, the maximum crack width and crack spacing of the six comparable test walls with shear span ratio of 4 (Wall C2, C6, M1, M2, M3 and M4) reported by Lu [6] are plotted against the secondary cracking index in Figure 8 and Figure 9, respectively. All the six walls had R6 horizontal reinforcement at 150 mm centers over whole wall height. Additional transverse reinforcement, consisting of R6 stirrups were placed at 60 mm centers in the wall ends over the lower 1.4 m of the

wall section in walls C6, M1, M2, M3 and M4 but not in wall C2. The average flexural crack spacing was estimated as the height over which the cracking extended up the wall divided by the number of the cracks at the wall edge and the crack width and the number of cracks were both measured at the lateral drift of 2.5%. The direct concrete tensile strength (f_{ct}) was calculated by Eq. 3 as per the *fib* model code [15]. The secondary index of each specimen is shown in Table 2.
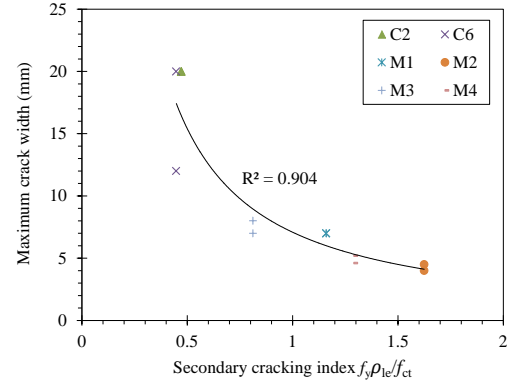

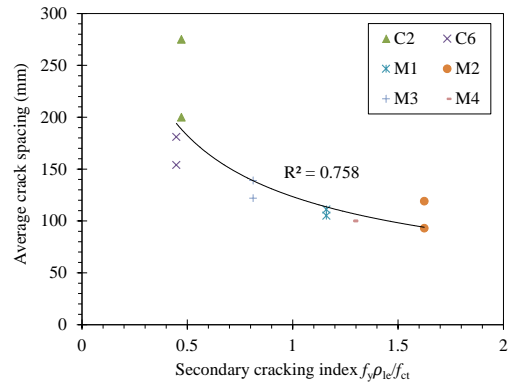**Figure 8: Maximum crack widths of test walls with shear span ratio of 4.**

Figure 9: Average crack spacing of test walls with shear span ratio of 4.

Table 2: Secondary cracking index of walls test by Lu [6].

Wall No.	$f_c^{'}$ (MPa)	f_y (MPa)	ρ_{le}	$f_y\rho_{le}/f_{ct}$
C2	34.5	300	0.50	0.47
C6	37.3	300	0.50	0.45
M1	37.1	387	1.00	1.16
M2	36.3	371	1.44	1.62
M3	36.3	371	0.72	0.81
M4	36.7	334	1.28	1.29

As shown in Figure 8 and Figure 9, as the secondary cracking index increased, the maximum crack width and average crack spacing both decreased, indicating that more secondary cracks occurred in the walls with a higher secondary cracking index, in line with what is proposed in Eq. 12. When comparing walls C2 and C6 that had a similar secondary cracking index but different stirrup spacing, the crack spacing of wall C6 was smaller than that of wall C2. However, the maximum crack width in wall C6 was similar with that of wall C2 and both walls were controlled by discrete cracking behaviour, indicating that closely placed stirrups could trigger more cracks but did not have a significant influence on the overall wall behaviour. It also should be noted that when the secondary cracking index was small, the trend was more obvious. For example, the maximum crack width decreased from 20 mm to 8 mm and the average crack spacing decreased from 275 mm to 122 mm when the index increased from 0.45 to 0.81. However, when the

index was larger, the trend started to flatten off as the reinforcement was already sufficient to ensure secondary cracks. Comparing the four test walls M1 to M4 for which the index ranged from 1.16 to 1.62, the maximum crack width varied from 4 mm to 8 mm and the average crack spacing only varied from 93 mm to 122 mm. This indicated that there was a threshold for the secondary cracking index to ensure well distributed secondary cracks. When using the proposed equation to calculate end zone reinforcement ratio, increasing the index can significantly improve the cracking behaviour of lightly reinforced concrete walls when the index is below the threshold (i.e. the wall is controlled by discrete cracks). However, when the index exceeds the threshold no further improvement can be achieved. From the test results of cracking behaviour presented in Figure 8 and Figure 9, the threshold to ensure well distributed secondary cracks was equal to a secondary cracking index of approximately 1.0, which further confirms the theory used to derive the secondary cracking criteria.

COMPARISON WITH CURRENT CODES

As presented previously, the minimum vertical reinforcement limits for RC walls vary substantially between different concrete design standards and are mostly based on engineering judgement. The proposed minimum vertical reinforcement requirements were derived from performance criteria and intended to address the deficiencies of existing requirements. To demonstrate their deficiencies, the minimum vertical reinforcement limits in current concrete standards were compared with the proposed requirements.

Margin of Safety for Moment Criterion

For RC walls designed to exhibit low or nominal ductility, lower minimum vertical reinforcement limits are usually required by concrete standards when compared to that for RC walls with higher ductility demands, as shown in Table 1. To examine whether these requirements can prevent non-ductile behaviour for RC walls during earthquakes, the likely range of margin of safety (M_n/M_{cr}) was calculated for the minimum vertical reinforcement limits in each concrete standard as well as the proposed distributed vertical reinforcement requirement in Eq. 9. The nominal flexural strength (M_n) was calculated based on a rectangular stress block using the specified concrete strength and lower characteristic yield strength of the reinforcement (as per NZS 3101:2006 and ACI 318-14) and cracking moment strength (M_{cr}) was calculated using Eq. 2.

Figure 10 shows the range of margin of safety (Ω) between cracking and nominal flexural strength for walls with each minimum vertical reinforcement limit listed in Table 1. The range of concrete compressive strength, reinforcement yield strength and axial load ratio were chosen to be 30-70 MPa, 300-600 MPa, and 0-20%, respectively. The wall section was assumed to have a length of 2800 mm and a thickness of 150 mm. Within the range of margin of safety, four cases were chosen to be plotted in Figure 10. The two outmost envelops represented the best and worst cases. The most favourable combination (case A) was found to have the lowest strength concrete and highest yield strength reinforcement, while the worst combination (case D) was found to have the highest strength concrete and lowest yield strength reinforcement. In addition, the two middle cases were chosen to both have a reinforcement yield strength of 500 MPa but have concrete compressive strengths of 30 MPa (case B) and 40 MPa (case C), respectively, to demonstrate the margin of safety trends.

As shown in Figure 10, the minimum vertical reinforcement requirements in current concrete design standards could not ensure a consistent margin of safety between cracking and nominal flexural strength. As shown in Figure 10-a, b and c, the margin of safety for RC walls with a fixed minimum vertical reinforcement content decreased as the concrete compressive strength increased and/or reinforcement yield strength decreased. The margin of safety of case B ($f_c' = 30$ MPa, $f_y = 500$ MPa) is lower than that of case A ($f_c' = 30$ MPa, $f_y = 600$ MPa), while the margin of safety of case C ($f_c' = 40$ MPa, $f_y = 500$ MPa) is lower than that of case B. NZS 3101:2006 (A2) and NZS 3101:2006 (A3 draft) both incorporate concrete and reinforcement strength in the equation for distributed reinforcement and so the margin of safety was consistent as the material strength changed. As shown in Figure 10-d, all the four cases with different material properties overlapped each other. However, no concrete standard currently incorporates axial load in the minimum vertical reinforcement limits. The margin of safety for all concrete standards increased when the axial load ratio increased from 0 to 20%, as shown in Figure 10-a, b, c and d. The axial load increases the cracking moment as it creates an initial pre-compression to the wall. However, the increase in the cracking moment is overshadowed by the increase in nominal flexural strength provided by the axial load. For the proposed equation based on moment criterion, consideration of concrete strength, reinforcement strength and the axial load ratio ensures that the margin of safety was consistent, as shown in Figure 10-e. These findings highlight the importance of accounting for both material properties and axial load when calculating minimum vertical reinforcement for walls designed for low or nominal ductility demands.

The lack of consideration of the critical parameters resulted in a large range in the margin of safety for RC walls with minimum vertical reinforcement in accordance with current concrete design standards. In several cases the extremely small margin of safety would likely result in non-ductile behaviour. As shown in Figure 10-a and b, for walls with a fixed minimum vertical reinforcement of 0.15% and 0.25% (as per ACI 318-14 and CSA A23.3-14), the margin of safety varied from 0.2 to 1.7 and from 0.4 to 1.8, respectively. These results indicated that there was a high risk that the cracking moment strength exceeds the nominal flexural capacity, resulting non-ductile behaviour. In particular, when the axial load ratio was below 3%, the margin of safety was extremely low and in most cases less than 1.0. It is recommended that for walls designed with a fixed distributed reinforcement ratio in seismic regions, the resulting margin of safety should be compulsorily checked to ensure that non-ductile behaviour is prevented.

Eurocode 8 and GB 50010-2010 require an end zone reinforcement ratio of 0.5%, and as a result the margin of safety of these walls was higher than that of walls with a fixed distributed minimum vertical reinforcement of 0.15% or 0.25%. However, the variation in the margin of safety was still large, ranging from 0.45 to 2.0, as shown in Figure 10-c. When the axial load ratio was larger than 5%, the margin of safety typically exceeded 1.5, indicating that the minimum vertical reinforcement limits in Eurocode 8 and GB 50010-2010 can ensure sufficient margin of safety when a reasonable axial load is applied. However, when the axial load was small, the margin of safety could still be less than 1.0, indicating that non-ductile behaviour is still possible for walls designed in accordance with Eurocode 8 and GB 50010-2010.
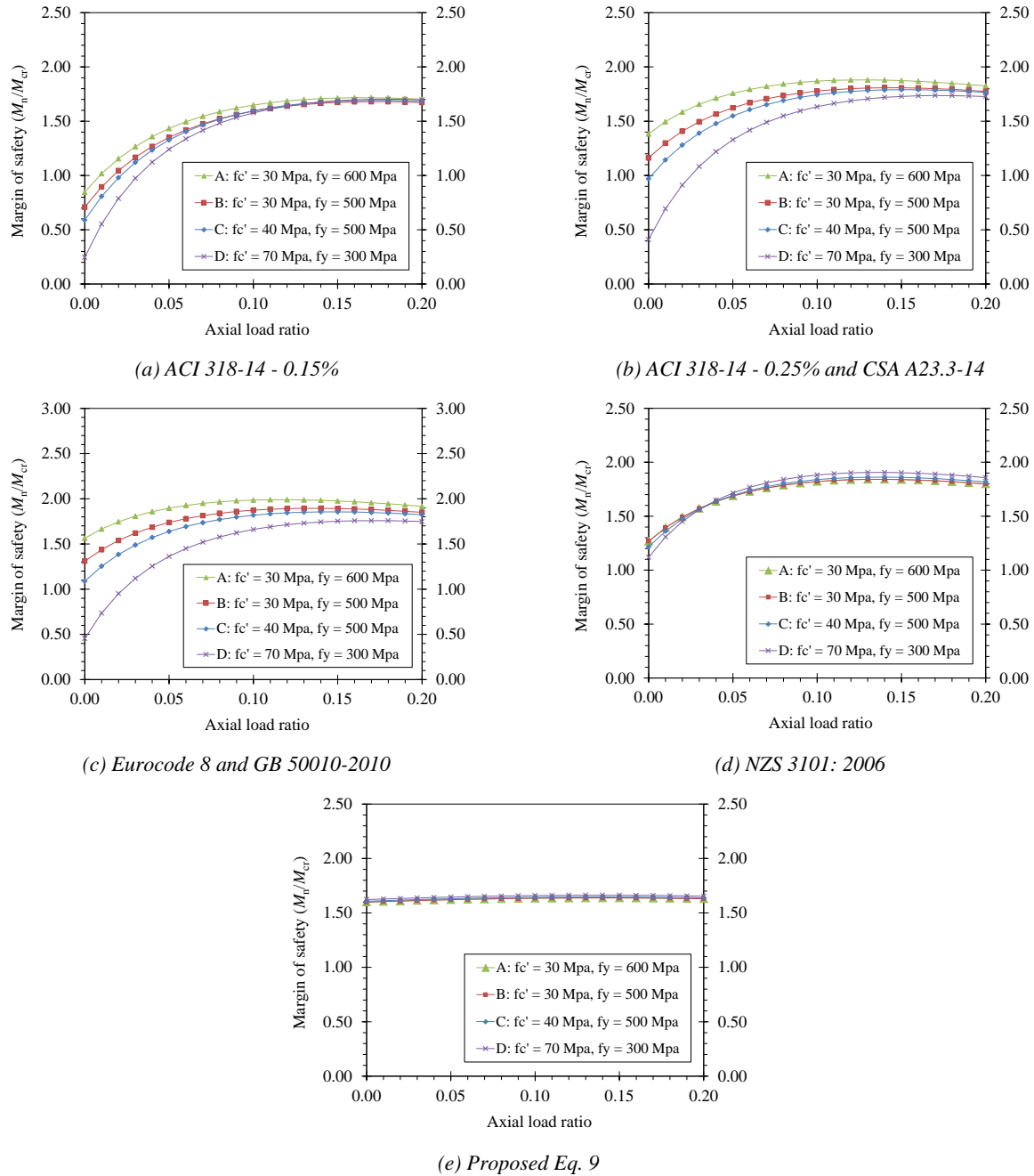
(a) ACI 318-14 - 0.15%

(b) ACI 318-14 - 0.25% and CSA A23.3-14

(c) Eurocode 8 and GB 50010-2010

(d) NZS 3101: 2006

(e) Proposed Eq. 9

Figure 10: Margin of safety M_n/M_{cr} for RC walls with different minimum vertical reinforcement requirements.

As previously discussed, NZS 3101: 2006 considers the material strengths and so the margin of safety for NZS 3101: 2006 walls was significantly more consistent than those designed in accordance with other design standards. As shown in Figure 10-d, the margin of safety between cracking and nominal flexural strength of NZS 3101: 2006 walls ranged from 1.1 to 1.8 and was typically larger than 1.5. The range from the calculation was fairly consistent with that from the test results reported by Lu et al. [4], where the margin between test flexural strength and test cracking strength of test walls with minimum vertical reinforcement in accordance with NZS 3101:2006 (A2) ranged from 1.43 to 1.87. The test results also implied that a margin of safety from 1.5 to 1.8 was sufficient to prevent non-ductile behaviour as all the test walls had reasonable ductility and drift capacity although well distributed secondary cracks did not occur. In some extreme cases, the margin of safety of the walls designed in accordance with NZS 3101:2006 could still be close to 1.0, which might not

be sufficient to prevent non-ductile behaviour when axial load is low and unexpected material strengths occur.

The safety factor when calculating the minimum vertical reinforcement as per the proposed Eq. 9 was assumed to be 1.6 to be consistent with the examples reported previously. The proposed equation considers all the three parameters and so the margin of safety was consistent at 1.6 for all the cases, as shown in Figure 10-e. The margin of safety between cracking and nominal flexural strength was calculated to be the same as the safety factor Ω, indicating that the simplified calculation in Eq. 9 can accurately estimate the minimum distributed vertical reinforcement ratio based on the moment criterion.

Index of Secondary Cracking Behaviour

For RC walls designed to exhibit ductility during earthquakes, some standards such as Eurocode 8, GB 50010-2010, CSA A23.3-14 and NZS 3101:2006 (A3 draft) require additional vertical reinforcement to be concentrated at the ends

of the walls, as shown in Table 1. To investigate whether they can ensure reasonable and consistent cracking behaviour, the likely range of the secondary cracking index was calculated for each end zone reinforcement requirement in the above concrete standards as well as the proposed requirements in Eq. 11.

Figure 11 plots the range of the secondary cracking index for each end zone reinforcement requirement with concrete compressive strength and reinforcement yield strength ranging from 30-70 MPa and 300-600 MPa, respectively. As shown in Figure 11-a and b, when the end zone vertical reinforcement content remained fixed, the secondary cracking index was smaller when using higher strength concrete or lower yield strength reinforcement, implying that the number of cracks of the walls was reduced. The secondary cracking index for an end zone vertical reinforcement ratio of 0.5% (as per Eurocode 8 for high ductility class) ranged from 0.3 to 1.05 with most cases below 1.0, which was the threshold for ensuring well distributed secondary cracks. For an average case with 40 MPa strength concrete and 500 MPa yield strength reinforcement, the secondary cracking index was calculated to be 0.7, not large enough to form well distributed secondary cracks. These results confirmed that a 0.5% end zone vertical reinforcement ratio cannot ensure well distributed cracks in RC walls. The GB 50010-2010 and CSA A23.3-14 standards both require 1.0% end zone reinforcement ratio when the end zone length was taken as $0.15l_w$. As the reinforcement ratio was doubled when compared to the Eurocode 8 requirements, the secondary cracking index also doubled ranging from 0.6 to 2.1. When the concrete strength was lower than 40 MPa, the secondary cracking index was typically larger than 1.0. The cases in which the index was below 1.0 only occurred when high strength concrete was combined with low yield strength reinforcement. For the average case with 40 MPa strength concrete and

500 MPa yield strength reinforcement, the secondary cracking index was 1.4, which was sufficient to ensure well distributed secondary cracks over the wall height. These findings were also consistent with the numerical model results of walls designed in accordance with minimum vertical reinforcement requirement in Eurocode 8, GB 50010-2010 and CSA A23.3-14 presented by Lu [6].

NZS 3101:2006 (A3 draft) and the proposed requirements use the same secondary crack criteria to develop the equations for end zone reinforcement. The detailed modification factors that account for dynamic loading, drying shrinkage of concrete and average long-term material strengths are also the same for both the proposed Eq. 12 and NZS 3101:2006 (A3 draft). The only difference between these two requirements is the equation used to define the concrete tensile strength. The proposed Eq. 12 used $0.3f_c'^{\frac{2}{3}}$ as recommended by *fib*, while NZS 3101:2006 (A3 draft) used $0.52\sqrt{f_c'}$, which was consistent with test data of supplied New Zealand concrete [16]. Therefore, when considering the corresponding definitions for concrete tensile strength, the secondary cracking index for both NZS 3101:2006 (A3 draft) and the proposed Eq. 12 should equal the equivalent λ calculated based on the combination of material modification factors. As shown in Figure 11-c and d, the secondary cracking index for the proposed requirement was equal to 1.0 for all combinations of material strengths, while the NZS 3101: 2006 (A3 draft) was equal to 0.96 for all combinations of material strengths. This confirmed that the minimum end zone reinforcement requirements in accordance with both NZS 3101:2006 (A3 draft) and the proposed requirement can consistently ensure that well distributed secondary cracks occur.
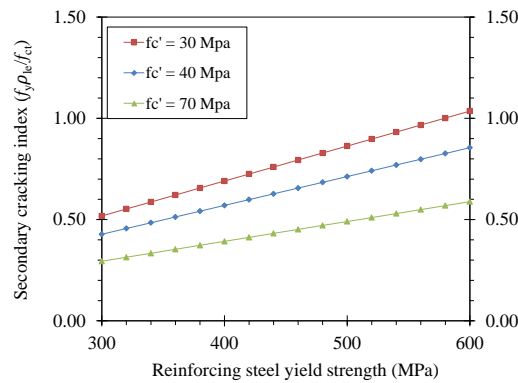
(a) Eurocode 8
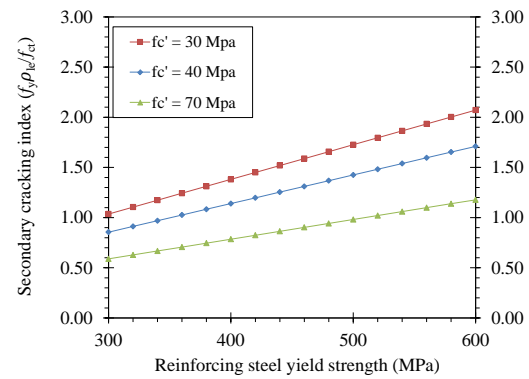
(b) GB 50010-2010/ CSA A23.3-14

(c) NZS 3101:2006 (A3 draft)

(d) Proposed Eq. 12

Figure 11: Secondary cracking index for each end zone reinforcement requirement.