ASSESSMENT OF MINIMUM VERTICAL REINFORCEMENT LIMITS FOR RC WALLS

R. S. Henry[1]

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ABSTRACT

During the 2010/2011 Canterbury earthquakes, several reinforced concrete (RC) walls in multi-storey buildings formed a single crack in the plastic hinge region as opposed to distributed cracking. In several cases the crack width that was required to accommodate the inelastic displacement of the building resulted in fracture of the vertical reinforcing steel. This type of failure is characteristic of RC members with low reinforcement contents, where the area of reinforcing steel is insufficient to develop the tension force required to form secondary cracks in the surrounding concrete. The minimum vertical reinforcement in RC walls was increased in NZS 3101:2006 with the equation for the minimum vertical reinforcement in beams also adopted for walls, despite differences in reinforcement arrangement and loading. A series of moment-curvature analyses were conducted for an example RC wall based on the Gallery Apartments building in Christchurch. The analysis results indicated that even when the NZS 3101:2006 minimum vertical reinforcement limit was satisfied for a known concrete strength, the wall was still susceptible to sudden failure unless a significant axial load was applied. Additionally, current equations for minimum reinforcement based on a sectional analysis approach do not adequately address the issues related to crack control and distribution of inelastic deformations in ductile walls.

1 INTRODUCTION

Assessments of buildings following the Canterbury earthquakes highlighted several examples of reinforced concrete (RC) walls that had formed a single crack in the plastic hinge region as opposed to the expected larger number of distributed cracks (Kam *et al.* 2011; Structural Engineering Society of New Zealand (SESOC) 2011b; Bull 2012). After breaking out the surrounding concrete it was found that the vertical reinforcing steel was often fractured due to the inelastic demand at the single crack location. This type of failure is characteristic of RC members with low vertical reinforcement contents and was also observed in buildings following the 1985 Chilean Earthquake (Wood 1989; Wood *et al*. 1991). As described by Paulay and Priestley (1992), if a section has insufficient reinforcement there is a danger that the probable cracking moment may exceed the section flexural strength resulting in a sudden non-ductile failure. If too little vertical reinforcement is used in walls, there is insufficient tension generated to replace the tensile resistance provided by the surrounding concrete after a crack forms, resulting in a reduced number of cracks in the critical moment region, large crack widths, and possible fracture of the reinforcing steel.

As part of the Canterbury Earthquakes Royal Commission (CERC) a detailed investigation was conducted into the performance of the Gallery Apartments building (Smith and England 2012). The RC walls in the Gallery Apartment building contained less vertical reinforcement than is required by current design standards and were observed to have formed a small number of cracks at the wall base. As highlighted in volume 2 of the CERC final report, the quantity of vertical reinforcement in the Gallery Apartment walls was insufficient to develop the tension force required to form secondary cracks (Canterbury Earthquakes Royal Commission 2012). Additionally, samples extracted from the Gallery Apartments building following the earthquakes indicated that the concrete strength was significantly higher than the specified concrete strength (Holmes Solutions 2011). The higher concrete strength further increased the likelihood of a single crack formation and fracture of vertical reinforcing steel.

A summary of the development of minimum reinforcement limits for both RC beams and RC walls is presented, followed by a series of moment-curvature analyses conducted to investigate the expected behaviour of one of the RC walls in the Gallery Apartment building. Additional analyses are presented that investigate the suitability of the minimum vertical reinforcement limits for walls specified in the current version of the New Zealand Concrete Structures Standard, NZS 3101:2006. Other factors that may have contributed to the observed behaviour of lightly reinforced concrete walls during the Canterbury earthquakes, including higher than specified concrete strength, loading history, and loading rate, were not considered within the scope of this investigation.

[1]*The University of Auckland, Auckland, New Zealand (Member).*

2 MINIMUM LONGITUDINAL/VERTICAL REINFORCEMENT LIMITS

Most design standards impose minimum and maximum limits on reinforcement for different types of concrete members. In addition to minimum required reinforcement to mitigate shrinkage and temperature effects, limits on reinforcement are also used to ensure that a ductile response is achieved. To ensure a ductile flexural response, minimum longitudinal or vertical reinforcement is required in concrete beams, columns, and walls.

2.1 RC Beams

The current version of the New Zealand Concrete Structures Standard, NZS 3101:2006, states that the area of longitudinal reinforcement in beams should satisfy Eq. 1 (clause 9.3.8.2.1).

$$A_s \geq \frac{\sqrt{f_c'}}{4f_y} b_w d \tag{1}$$

where A_s is the area of flexural tension reinforcement, f_c' is the specified concrete compressive strength, f_y is the yield strength of the longitudinal reinforcement, b_w is the width of the beam web, and d is the effective beam depth.

The origins of Eq. 1 can be traced back through early concrete design codes. A previous version of Eq. 1 that was intended to balance the cracking moment with a cracked flexural strength at a steel stress of $2f_y/3$ was later modified to account for variable concrete strength with an assumed modulus of rupture equal to $0.62\sqrt{f_c'}$ MPa (ACI 363R-84 1984). It has been suggested that Eq. 1 result in a nominal flexural strength between 1.5 to 2 times greater than the probable cracking moment of a rectangular beam (Paulay and Priestley 1992; Wang *et al.* 2007; Wight and MacGregor 2009).

The cracking moment (M_{crack}) of an unreinforced rectangular beam with a width b_w, height h, and a modulus of rupture f_r is shown in Eq. 2a. The nominal flexural strength (M_n) for a RC beam is calculated using Eq. 2b, where jd is the lever arm between the tension and compression resultant forces. For a beam with minimum longitudinal reinforcement the lever arm jd is likely to be approximately $0.95d$, and beam height h can be approximated as $1.1d$. Using these assumptions, Eq. 2a and Eq. 2b can be combined to form Eq. 2c., and then rearranged to give Eq. 2d.

$$M_{crack} = \frac{b_w h^2}{6} f_r \tag{2a}$$

$$M_n = A_s f_y jd \tag{2b}$$

$$\frac{M_n}{M_{crack}} = \frac{6A_s f_y jd}{b_w h^2 f_r} = \frac{6A_s f_y 0.95d}{b_w (1.1d)^2 f_r} = \frac{4.71 A_s f_y}{b_w d f_r} \tag{2c}$$

$$\therefore A_s = \frac{M_n}{M_{crack}} \frac{b_w d}{4.71 f_y} f_r \tag{2d}$$

By equating Eq. 1 and Eq. 2d, the ratio of nominal flexural strength to cracking moment (M_n/M_{crack}) for a rectangular beam with minimum longitudinal reinforcement would be equal to 1.90 when a modulus of rupture of $0.62\sqrt{f_c'}$ is applied, or equal to 1.18 when an upper characteristic modulus of rupture of $\sqrt{f_c'}$ is applied.

2.2 RC Walls

Versions of NZS 3101 Concrete Structures Standard prior to 2006 specified a minimum ratio of wall vertical reinforcement to gross section area, ρ_t, as shown in Eq. 3.

$$\rho_t \geq \frac{0.7}{f_y} \tag{3}$$

The origins of Eq. 3 are associated with requirements for shrinkage and temperature effects, and resulted in a minimum content of vertical reinforcement between 0.14 - 0.24% depending on the reinforcing steel grade. Interestingly, by analysing the results of 37 RC wall tests completed between 1950 and 1984, Wood (1989) concluded that walls with vertical reinforcement contents less than 1% were susceptible to premature fracture of reinforcing steel. Perhaps in response to Wood's analysis, the ACI 318-95 concrete deign code introduced an additional requirement that special structural walls designed for seismic applications should have a vertical reinforcement content greater than 0.25%.

In the NZS 3101:2006 revision, the minimum vertical reinforcement in walls was increased with the adoption of Eq. 4 (clause 11.3.11.3).

$$\rho_n \geq \frac{\sqrt{f_c'}}{4f_y} \tag{4}$$

where ρ_n is the ratio of vertical reinforcement area to the gross section area (A_t/A_g).

On the surface it appears as if the equation previously developed for RC beams (Eq. 1) has simply been applied to walls. However, it should be noted that there are distinct differences between walls and beams that could lead to inaccuracies when Eq. 4 is used, notably:

- Eq. 4 refers to the total vertical reinforcement in walls, A_t, compared with the area of tension reinforcement, A_s, in beams in Eq. 1;
- Vertical reinforcement is typically distributed along the wall length compared to lumped reinforcement at the top and bottom of beam sections;
- Wall sections are typically more slender than beams;
- Walls often have significant axial loads;
- Scale effects influence the flexural tensile strength of concrete and typical wall lengths are significantly larger than typical beam depths.

These listed differences between walls and beams would alter the assumptions used to derive Eq. 1 and may result in a reduced margin of safety between the flexural and cracking strength of a typical wall section.

Following the observations of RC wall behaviour during the Christchurch earthquakes, the Structural Engineering Society of New Zealand (SESOC) issued interim design guidelines that included Eq. 5 for the minimum vertical reinforcement in RC walls (2011a).

$$\rho_n \geq \frac{0.4\sqrt{f_c'}}{f_y} \tag{5}$$

Eq. 5 is modified from Eq. 4 based on the assumption that the actual concrete strength is 2.5 times greater than the specific concrete strength. The intention was that the 2.5 factor would account for mean concrete strengths, use of concrete strength greater than specified, additional strength gain over time, and dynamic strength enhancement. This issue of higher concrete strength was further highlighted in the CERC final report (2012) and is currently being investigated by the New Zealand concrete industry.

3 MOMENT-CURVATURE ANALYSIS

A series of analyses were conducted to investigate the behaviour of RC walls with minimum vertical reinforcement. Moment-curvature sectional analyses were performed using Respone-2000 software (Bentz and Collins 2001). Response-2000 also incorporates Modified Compression Field Theory (MCFT) to extend the moment-curvature results to a simple 2D member analysis. Analyses were first performed based on one of the walls in the Gallery Apartment building, after which the reinforcing contents were modified to investigate the current NZS 3101:2006 provisions.

3.1 Material properties

The concrete material properties entering into Response-2000 were based on either the specified or measured concrete compressive strength, f_c', with default base curve, and compression and tension stiffening. When confinement was provided in the wall ends through the use of transverse reinforcement, the compressive stress-strain backbone response was defined using procedures proposed by Mander *et al.* (1988). The flexural tensile strength (or modulus of rupture) assumed for the concrete was selected to represent either a realistic average or alternatively an upper characteristic estimate of the wall cracking strength rather than lower characteristic properties that are specified in design standards. The mean concrete tensile strength, f_{ct}, can be estimated using Eq. 6 (Fédération Internationale du Béton (fib) 2012), with the upper characteristic value obtained by multiplying the mean by a factor of 1.3.

$$f_{ct} = 0.3(f_c')^{2/3} \tag{6}$$

As mentioned earlier, there is a notable scale effect that influences the flexural tensile strength, $f_{ct,fl}$, of concrete members (Bažant 1984; Fenwick and Dickinson 1987; Carpinter 1989). As the depth of the member increases, the ratio of the flexural tensile strength to direct tensile strength decreases. This scale effect can be accounted for by using Eq. 7 that are derived from principles of fracture mechanics (Fédération Internationale du Béton (fib) 2012). It should be noted that the calculated α_f values are in agreement with the values listed in Table C5.1 of the NZS 3101:2006 commentary and for typical wall lengths results in flexural tensile strengths only slightly greater than the direct tensile strength.

$$f_{ct,fl} = \frac{f_{ct}}{\alpha_f} \tag{7}$$

$$where:\ \alpha_f = \frac{0.06h^{0.7}}{1+0.06h^{0.7}} \tag{7a}$$

The concrete flexural tensile strength used in the analyses to calculate the cracking moment was calculated using Eqns. 6 and 7. For a concrete compressive strength of 30 MPa this resulted in a mean flexural tensile strength of 3.03 MPa and an upper characteristic flexural tensile strength of 3.94 MPa.

The properties of the reinforcing steel were also varied for different analyses to represent the use of either lower characteristic or average values. Grade 500 reinforcing steel was used throughout the analyses, consistent with that used in the walls of the Gallery Apartments building. The lower characteristic properties used included a yield strength, f_y, of 500 MPa, a ratio of ultimate strength to yield strength (f_u/f_y) of 1.15, and a maximum elongation of 10%. The average properties specified were based on previous testing of G500 reinforcing steel, including yield strength, f_y, of 560 MPa, an ultimate strength, f_u, of 690 MPa, and a maximum elongation of 12.9%.

3.2 Gallery Apartments building

As described earlier, several RC walls in the Gallery Apartments building were observed to have formed a small number of cracks at the wall base that resulted in fracture of the vertical reinforcement. The seismic performance of the Gallery Apartments building was analysed in detail as part of the CERC (Canterbury Earthquakes Royal Commission 2012; Smith and England 2012). One of the critically damaged RC walls on grid-F was analysed to investigate the expected behaviour using moment-curvature analysis. The grid-F wall had a length of 4300 mm, a thickness of 325 mm, two layers of DH12 vertical reinforcement at 460 mm centres, and DH12 closed stirrup transverse reinforcement at 400 mm centres. Although the building was designed assuming a limited ductile response, detailing of the RC walls was only consistent with requirements for a nominally ductile response with no transverse confinement reinforcement in the wall ends (Smith and England 2012). The specified 28-day concrete strength of the RC walls was 30 MPa, however, tests performed on two concrete cores extracted from the building indicated compressive strengths of 46.5 MPa and 56.0 MPa (Holmes Solutions 2011). The axial load on the grid-F wall was calculated by Smith and England (2012) to be 2,250 kN, just over half of which (1,308 kN) is attributed to the self-weight of the wall. The ratio of vertical reinforcement in the grid-F wall, ρ_n, is equal to 0.160%, slightly above the 0.140% minimum required by NZS 3101:1995 (Eq. 3) but below the 0.274% minimum required by NZS 3101:2006 (Eq. 4).

The results from the moment-curvature analysis performed on the grid-F wall from the Gallery Apartments building are summarised in

Table 1, where M_{crack} is the cracking moment, M_{yield} is flexural strength when the vertical reinforcement steel yields, and M_{max} is the maximum flexural strength immediately prior to failure. The analysis was completed for combinations of concrete compressive strength, concrete tensile strength, reinforcing steel properties, and axial load. The moment-curvature results in

Table 1 highlight the vulnerability of the grid-F wall to sudden failure and loss of strength, confirming the inadequacy of minimum vertical reinforcement provisions based solely on temperature and shrinkage effects. Of the 10 different analysis options considered, half resulted in a section yield moment less than the cracking moment, indicating that the wall cannot retain its lateral resistance after the formation of a crack at the wall base. This drop in strength following the crack formation can be seen in the moment-curvature analysis results for wall G2, which are plotted in Figure 1.

Table 1 – Summary of moment-curvature results for the Gallery Apartments grid-F wall

Ref.	Concrete properties		Reo. properties	Axial load (kN)	M_{yield} (kNm)	M_{max} (kNm)	M_{crack} (kNm)	M_{yield}/M_{crack}	M_{max}/M_{crack}
	f_c' (MPa)	$f_{ct,fl}$ (MPa)							
G1	30.0	3.03 [Mean]	LC	0	2221.4	2638.7	3039.3	0.73	0.87
G2	30.0	3.94 [UC]	LC	0	2241.5	2623.5	3951.1	0.57	0.66
G3	30.0	3.03 [Mean]	LC	2250 [5.4%]	6130.1	6786.6	4651.8	1.32	1.46
G4	30.0	3.94 [UC]	LC	2250 [5.4%]	6152.4	6786.8	5563.6	1.11	1.22
G5	30.0	3.03 [Mean]	Average	2250 [5.4%]	6405.8	7059.4	4651.8	1.38	1.52
G6	30.0	3.94 [UC]	Average	2250 [5.4%]	6435.1	7059.6	5563.6	1.16	1.27
G7	51.3*	4.34 [Mean]	Average	2250 [3.1%]	6528.2	7449.6	5958.6	1.10	1.25
G8	51.3*	5.64 [UC]	Average	2250 [3.1%]	6554.3	7452.4	7262.4	0.90	1.03
G9	51.3*	4.34 [Mean]	Average	0	2506.7	3145.3	4346.1	0.58	0.72
G10	51.3*	5.64 [UC]	Average	0	2527.3	3134.6	5649.9	0.45	0.55

*51.3 MPa is the mean concrete strength of the two extracted core samples.
LC = lower-characteristic; UC = upper-characteristic.

When considering the measured concrete strength and the average reinforcing steel properties, the moment-curvature results vary from M_{yield}/M_{crack} ratios of 0.45-1.10 depending on the assumed concrete tensile strength and whether the axial load can be relied on. These results confirm the observed behaviour of the grid-F wall with a single crack formation and ruptured reinforcing steel and the increased vulnerability of walls when the actual concrete strength is considerably greater than specified strength.

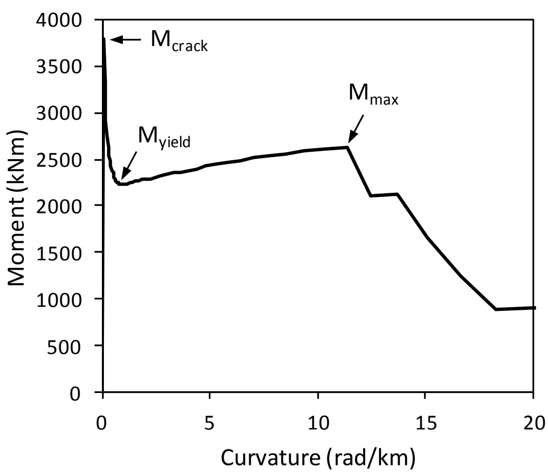


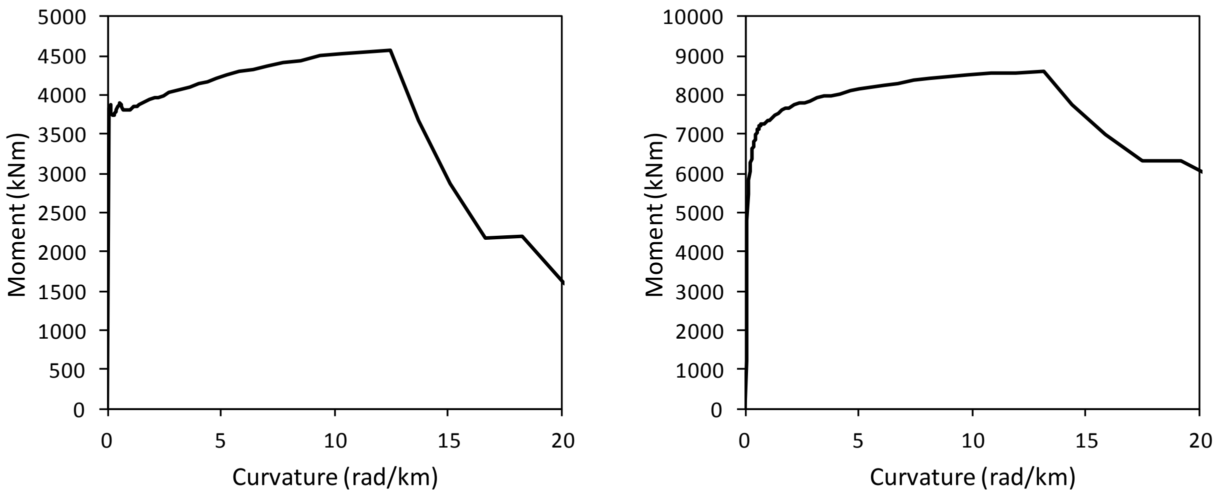
Figure 1: Moment-curvature response for the Gallery Apartments grid-F wall G2 analysis.

3.3 NZS 3101:2006 Standard

Moment-curvature analysis was also conducted on a modified wall design using the dimensions of the Gallery Apartments grid-F wall with vertical reinforcing consistent with the current minimum vertical reinforcement requirements in NZS 31010:2006 (Eq. 4) and the specified concrete compressive strength of 30 MPa. For these analyses conservative material properties were used, including the upper-characteristic concrete tensile strength ($f_{ct\,fl}$ = 3.94 MPa) and the lower characteristic grade 500 reinforcing steel properties (f_y, = 500 MPa, f_u/f_y = 1.15, and ε_{su} = 10%). In order to satisfy the NZS3101:2006 minimum vertical reinforcement limits the DH-12 bars in the as-built grid-F walls were replaced with DH-16 bars. The larger bar diameter resulted in a vertical reinforcement content of 0.288%, slightly greater that the 0.274% minimum calculated from Eq. 4.

The resulting moment-curvature response for the modified grid-F wall with DH-16 bars is shown in Figure 2 for analyses with both no axial load applied to the wall (D1) and an axial load ratio ($N/A_g f_c'$) equal to 5% (D2). When no axial load is applied the wall exhibits a borderline non-ductile response losing some strength immediately after cracking and rupture of the vertical reinforcement initiating at a flexural strength only 15% greater than the cracking strength. This analysis with no axial load highlights the differences between beam and walls. As described earlier a beam with minimum reinforcing and no axial load would be expected to have a significant margin of safety, with the nominal flexural strength between 1.5-2 times greater than the probable cracking moment. The moment-curvature analysis with an axial load ratio of 5% indicates a more desirable behaviour with no sudden loss of strength following cracking and significant strain hardening prior to rupture of the vertical reinforcement. The axial load creates an initial pre-compression to the wall that increased the moment required to cause cracking. However, the increase in the cracking moment is overshadowed by the increase in ultimate flexural strength provided by the axial load. Interestingly, the positive effect of the axial load compensates for the reduced margin of safety that was observed in the moment-curvature analysis of the wall with no axial load.

The moment-curvature analysis for the modified grid-F wall with distributed DH16 bars was conducted for axial load ratios of 0, 5, 10, 15, and 20%. The results of these 5 analyses with different axial loads are summarised in Table 2 and the margin of safety between the cracking moment and flexural strength is also plotted in Figure 3. The trend that was observed in the first two analyses plotted in Figure 2 continues with the larger increase in axial load. When an axial load ratio in excess of 10% is applied, the margin of safety between the cracking moment and ultimate flexural strength is similar to that of an RC beam. The trend indicated in Figure 3 highlights the importance of accounting for the axial load when considering the flexural response of walls. However, axial load ratios of less than 5% are common for walls with minimum vertical reinforcement and so walls constructed to the NZS 3101:2006 Standard could still be vulnerable to non-ductile failure.



(a) D1: No axial load

(b) D2: 5% axial load

Figure 2: Moment-curvature responses for the NZS 3101:2006 walls with distributed vertical reinforcement.

Table 2 – Summary of moment-curvature results for the walls with distributed vertical reinforcement conforming to NZS 3101:2006

Ref.	**Axial load ratio**	**M_{yield} (kNm)**	**M_{max} (kNm)**	**M_{crack} (kNm)**	**M_{yield}/M_{crack}**	**M_{max}/M_{crack}**
D1	0	3801	4546	3946	0.96	1.15
D2	5%	7256	8613	5448	1.33	1.58
D3	10%	10394	12391	6951	1.50	1.78
D4	15%	13335	15992	8453	1.58	1.89
D5	20%	16252	19585	9955	1.63	1.97

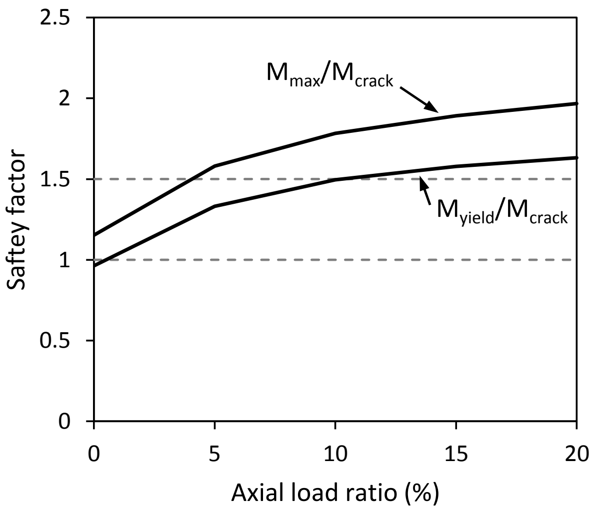


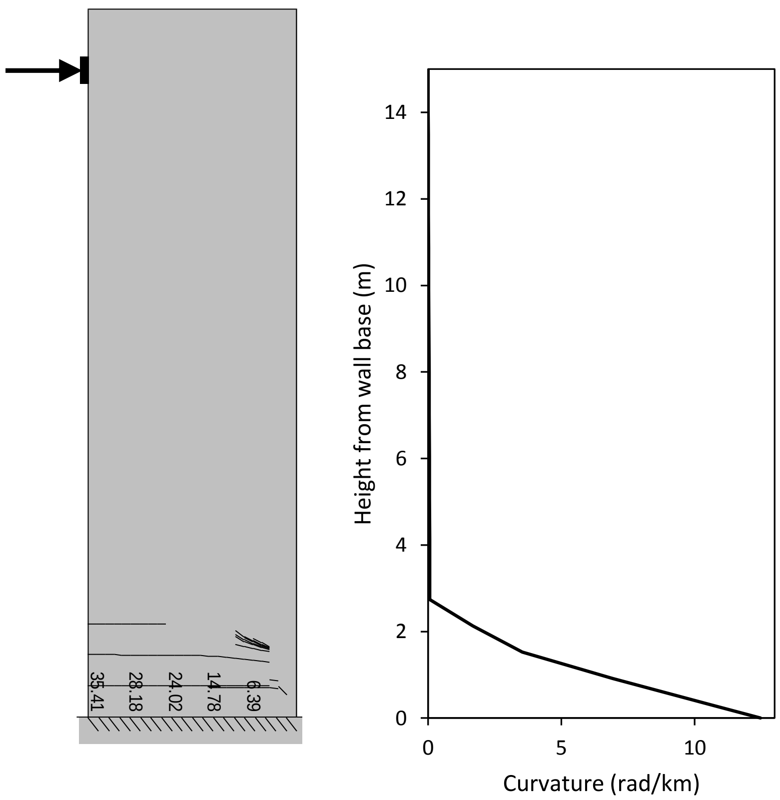
Figure 3: Safety factors for different axial load ratios.

3.4 Plastic hinge length

Assessment of the margin of safety between the cracking moment and ultimate flexural strength of a wall section may prevent a sudden undesirable failure, but it does not explicitly address the distribution of inelastic deformations within the wall. A RC wall must form a plastic hinge with sufficient ductility to provide a suitable seismic load resisting system for a multi-storey building. The ductility of the plastic hinge is severely compromised when deformations are concentrated at a few large cracks as opposed to a large number of distributed cracks.

The member analysis function of Response-2000 was utilised to investigate the crack distribution and spread of inelastic deformations in the modified grid-F wall that satisfied the current NZS 3101:2006 vertical reinforcement limits. The seismic load was represented by a concentrated lateral force applied at an effective wall height of 15 m with a fixed constraint at the wall base. The crack pattern and curvature distribution up the height of the wall that were calculated for the analysis of wall D1 (no axial load ratio) are shown in Figure 4.

The crack pattern does indicate some secondary crack formation, but the deformation is predominantly concentrated at the initial crack at the wall base which opened up to a 35 mm width before the vertical reinforcing steel ruptured. The curvature distribution up the wall height confirms the concentration of inelastic deformation in a few cracks at the wall base, with all but the lower 3 m height of the wall predicted to remain elastic (uncracked).



(a) Member crack pattern (annotated with critical crack widths)

(b) Curvature distribution

Figure 4: Member analysis results for wall D1 (no axial load ratio).

Member analysis was conducted for all of the walls with distributed vertical reinforcement conforming to NZS 3101:2006 (D1-D5). An equivalent plastic hinge length was calculated for each wall by equating the area under the plastic curvature distribution to an equivalent rectangle with the same peak curvature. This procedure is the same as that used to derive the equivalent plastic hinge length and curvature limits in NZS 3101:2006, as discussed by Fenwick and Dhakal

(2007). The ultimate lateral drift calculated from the failure displacement at the top of the wall and the equivalent plastic hinge lengths calculated from the wall analyses are given in Table 3. The ultimate lateral drift increased from 1.70% to 2.10% as the axial load ratio was increased from 0-20%. These drift capacities are less than the allowable drift limits for ductile buildings, highlighting the possibility of premature failure of the lightly reinforced walls in limited-ductile or ductile structures. The calculated equivalent plastic hinge lengths assumed by NZS 3101:2006 would be $l_w/2$, or 2.15 m, approximately twice the actual equivalent plastic hinge length calculated from the member analysis.

Table 3 – Equivalent plastic hinge lengths for the walls with distributed vertical reinforcement conforming to NZS 3101:2006

Ref.	Axial load ratio	Ultimate lateral drift (%)	Equivalent plastic hinge length	
			Calculated (mm)	% of NZS 3101
D1	0	1.70	1127	52
D2	5%	1.72	1132	53
D3	10%	1.84	1177	55
D4	15%	1.87	1207	56
D5	20%	2.10	1224	57

3.5 Discussion and Recommendations

3.5.1 *Minimum vertical reinforcement limits*

The moment-curvature analysis of the as-built Gallery Apartment grid-F wall highlighted the vulnerability of walls constructed with minimum vertical reinforcement prior to the adoption of NZS 3101:2006. If yielding is expected in a wall with minimum vertical reinforcement based on shrinkage and temperature requirements there is a strong possibility that a sudden failure could occur due to the probable cracking moment being greater than the flexural strength of the cracked section. Furthermore the equation adopted by NZS 3101:2006 for minimum vertical reinforcement in walls may not be suitable for all applications. To achieve the equivalent margin of safety between the nominal flexural strength and cracking moment of a beam, a wall would need to be subjected to an axial load of approximately 10% of the section capacity. However, the reinforcement arrangement and loading of walls are a significant deviation for the assumptions that were originally used to derive Eq. 1 and it is not desirable to rely on axial load to achieve a ductile response.

3.5.2 *Spread of plasticity*

Although the current equations for minimum longitudinal and vertical reinforcement may prevent a sudden failure caused by an abrupt drop in section flexural strength, the ductility of the member is not specifically addressed. Even when the sectional analysis indicated a significant margin of safety between the cracking moment and ultimate flexural strength, the member analysis still showed a concentration of inelastic deformations over a relatively short height of the wall. For all analyses a large crack was predicted to open up at the wall base which eventually led to fracture of the vertical reinforcement. This concentration of inelastic deformations in a single crack has significant consequences with regard to the procedures used in NZS 3101:2006 to estimate allowable curvature limits. Alternative requirements for minimum vertical reinforcement may be required if a wall is intended to provide the lateral resistance in a multi-storey building designed to exhibit a limited ductile or ductile seismic response. It should be noted that the member analysis conducted in Response has not yet been verified against experimental data and a series of tests and more sophisticated analyses are required to confirm these requirements.

The approach recommended by CERC (2012) is to investigate the reinforcement required to initiate secondary flexural cracks in walls. The minimum vertical reinforcement calculated from Eq. 4 can be rearranged to investigate the direct tensile stress induced in the surrounding concrete, as shown below in Eq. 8. When considering the direct tension force required to initiate a secondary crack, Eq. 8 implies that a maximum direct tensile stress of $0.25\sqrt{f_c'}$ can be overcome in the surrounding concrete. This tensile stress is approximately half of the estimated direct tensile strength of concrete which implies that secondary cracks would be unlikely to form when minimum vertical reinforcement is used (Mindess *et al.* 2003; Fédération Internationale du Béton (fib) 2012). Additionally, the effective area of concrete in tension that should be used to determine the criteria for secondary cracks needs to be considered along with the stress concentrations that may initial a crack.

$$A_{steel} f_y \geq 0.25\sqrt{f_c'} A_{concrete} \tag{8}$$

3.5.3 *Distribution of reinforcement*

The analyses presented were conducted with evenly distributed bars to satisfy the minimum NZS 3101:2006 minimum vertical reinforcement requirements (Eq. 4). Distributed vertical reinforcement in RC walls is common practice in New Zealand and was previously recommended to avoid undesirable shear failure modes (Park and Paulay 1975; Paulay and Priestley 1992). In the published interim design guidelines, SESOC recommended that vertical reinforcing steel should instead be lumped at the ends of the wall with minimum reinforcement along the wall web (Structural

Engineering Society of New Zealand (SESOC) 2011a). It was suggested that increasing the reinforcement in the ends of the wall would increase the tension force and thus increase the chances of forming secondary cracks. However, research by Dai (2012) found that concentrating too much vertical reinforcement in the boundary regions can leave the web region vulnerable to undesirable failures that reduce the wall ductility. Additionally, sufficient distributed reinforcement is required in the web region to encourage the propagation of secondary cracks within the section (Fenwick 1988).

Further research is required to clarify the impact of reinforcement distribution on RC wall behaviour with consideration to different wall typologies. For example, recommendations suitable for multi-storey ductile RC walls may be different to that suitable for low-rise nominally ductile RC walls. Additionally, recommendations for distribution of reinforcement may need to consider the magnitude of shear stresses on the wall and vertical reinforcing contents.

3.5.4 *Other issues*

The moment-curvature analysis indicated that increasing the axial load improved the margin of safety between the nominal flexural strength and the probable cracking moment. However, reliance on axial load is not sufficient to ensure a ductile response is achieved in RC walls. The analysis results indicate that even for walls with modest axial loads, a significant portion of the flexural capacity in walls with minimum vertical reinforcement is attributed to the axial load. This reliance on axial load may result in low hysteretic energy dissipation for some walls. Additionally, the analysis indicated that the spread of plasticity was not significantly affected by the magnitude of axial load, with only a small increase in the equivalent plastic hinge length. This is because the axial load does not influence the criteria for the formation of secondary cracks.

The crack distribution in RC walls with minimum vertical reinforcement also has an influence on the section properties used to conduct analysis. The concentration of deformations in only a small number of cracks at the wall base means that the upper stories of the wall remain uncracked. The use of typical cracked section properties to analyse lightly reinforced concrete walls will overestimate the fundamental period and could result in an inaccurate calculation of seismic design forces. Further investigation is required to determine appropriate section properties for use in analysing RC walls with minimum vertical reinforcement.

4 CONCLUSIONS

An investigation was conducted to examine the current minimum vertical reinforcement limits set out in the New Zealand Concrete Structures Standard NZS 3101:2006. The development of the current equations was presented in addition to a series of moment-curvature and member analyses of an example multi-storey RC wall. The following conclusions were drawn from the results of this investigation:

1. Current limits for minimum longitudinal in beams and vertical reinforcement in walls are intended to prevent a sudden failure by ensuring that the nominal flexural capacity of a section exceeds the probable cracking moment.
2. The required minimum vertical reinforcement in walls increased significantly in the NZS 3101:2006 standard with the adoption of a similar equation to that used for RC beams.
3. The vertical reinforcement content in the RC walls of the Gallery Apartments building was insufficient to ensure a ductile seismic response with distributed cracking at the wall base.
4. Walls designed with minimum vertical reinforcement in accordance with NZS 3101:2006 are reliant on a significant axial load acting to avoid the possibility of a sudden loss in strength after cracking.
5. A sectional analysis approach and the current minimum vertical reinforcement limits in NZS 3101:2006 do not ensure that inelastic deformations are well distributed with a large number of cracks at the base of the wall.
6. Further research, and in particular experimental testing, is required to investigate the performance of lightly reinforced RC walls and to determine suitable vertical reinforcement contents and distribution in multi-storey RC walls.

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