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A database for investigating NZS3101 structural wall provisions

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ABSTRACT:

During the 2010/2011 Canterbury Earthquakes several reinforced concrete (RC) walls in modern buildings with ductile detailing performed poorly and exhibited unexpected failure modes (e.g., local buckling of longitudinal reinforcement and global buckling of the wall section out-of-plane). These observations led to a number of revisions to NZS3101:2006 (A2) related to minimum reinforcement, transverse reinforcement ties, and axial load limitations. Due to the urgent need to make these changes despite the lack of availability of a comprehensive study to provide guidance, these amendments were not always verified with a robust set of experimental results. This paper summarises and assesses a number of these amendments through a newly created database that focuses on ductile, rectangular walls with particular attention paid to end region detailing, axial load conditions, and the response to uni-axial, reversed-cyclic, pseudo-static loading. Based on analyses conducted using the database, a reduced bar slenderness ratio (defined as the ratio of unsupported bar length to bar diameter) of 5 is recommended for walls with low axial loads ($<0.1A_gf'_c$), and a minimum transverse reinforcement ratio of 0.4% in the end region is suggested to reduce the likelihood of concrete crushing in walls designed with a limited ductile or ductile plastic hinge region. Preliminary assessment of the influence of confinement depth and web anti-buckling ties on the ultimate state behaviour of walls indicates that additional research is required on these topics.

1 INTRODUCTION

Reinforced concrete (RC) structural walls are often used in buildings to resist lateral loading associated with earthquakes and wind. Observed damage from the 2010/2011 Canterbury earthquakes has indicated that several walls performed poorly during these earthquakes, exhibiting unexpected failure modes such as crushing of concrete in compression (Fig. 1a), local buckling of longitudinal reinforcement in the wall web and end regions (Fig. 1b) and global buckling of the wall section out-of-plane (Fig. 1c) (Elwood 2013; Sritharan et al. 2014). In this paper local buckling refers to localised instability of individual reinforcement while global buckling refers to instability of walls on a sectional level. Similarly poor performance of walls has been observed as a result of the 2010 Maule (Chile) earthquake (Wallace et al. 2012).

Although the poor performance of RC walls in these earthquakes did not result in structural collapse in most instances, many buildings were severely damaged, exhibiting unexpected failure modes and may have collapsed during longer ground motion shaking. Investigation of the wall behaviour highlighted many examples where fundamental engineering principles had not been followed, in addition to potential shortcomings of the provisions in the New Zealand Concrete Structures Standard, NZS3101:2006 (A2). To address these issues, the Canterbury Earthquakes Royal Commission (CERC) released a series of wide-scoping recommendations relating to building performance during the earthquakes (CERC 2012) and the Structural Engineering Society New Zealand (SESOC) released detailed design guidance as an interim measure until design standards were amended or revised (SESOC 2013). Given the immediate need to issue revised design guidelines, these new recommendations were based on limited available research and professional judgement. When robust research and evidence was lacking the recommendations were often deliberately conservative in nature. Many of these recommendations are now incorporated into the upcoming NZS3101:2006 amendment 3 (A3) (2015) which will be published later this year. This paper summarizes these NZS3101 amendments and other relevant CERC and SESOC recommendations and assesses some of these provisions through the use of a newly-created database of RC wall tests from the last 30 years.

(a) (b) (c)

Figure 1: (a) concrete crushing in compression, (b) vertical reinforcement buckling at the end region, (c) out-of-plane global web buckling (Elwood 2013; Sritharan et al. 2014).

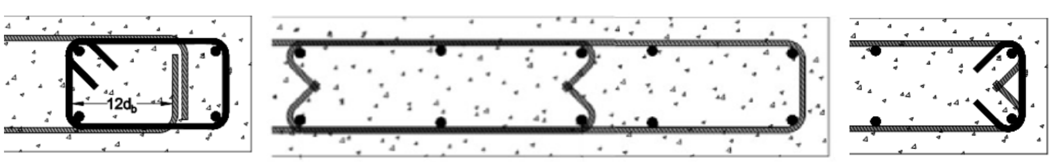
2 SUMMARY AND DISCUSSION OF RECOMMENDATIONS

A summary of new or amended provisions in NZS3101:2006 (A3) (2015), relative to NZS3101:2006 (A2) (2008), as well as of recommendations in SESOC (2013) and CERC (2012) related to non-coupled, doubly-reinforced RC walls (i.e. walls with two layers of reinforcement) is provided below. Although NZS3101:2006 (A3) has not been approved or published, significant changes are not expected and publication is expected in late 2015.

2.1 Confinement for anchorage

Observations from the Canterbury earthquakes highlighted several poor examples of horizontal reinforcement anchorage, such as 90-degree bends anchored in cover concrete (outside the confined core). As has been observed in other earthquakes, the 90-degree hooks started to open up and pull through when the cover concrete spalled. This loss of anchorage made the horizontal reinforcement ineffective in resisting shear actions and increased the likelihood of buckling of longitudinal reinforcement in the web of the wall.

The anchorage requirements for horizontal reinforcement in NZS3101:2006 (A3) were clarified as suggested in the SESOC interim design guidance (2013). The solutions that were considered to comply with the anchorage requirements in the standard are represented in Figure 2. It is suggested that 90-degree horizontal shear reinforcement hooks should be located inside of the confined concrete core (in the form of closed stirrup cages) (Fig. 2a), where concrete is less prone to crushing and spalling. Alternatively, it is permitted to use U-bars spliced to web horizontal reinforcement with 135-degree hooks around the longitudinal web reinforcement (Fig. 2b). It is also permitted to detail web horizontal reinforcement with 135-degree hooks that are located at the end of the walls and are anchored into the concrete core region and confined with an additional tie (Fig. 2c).



(a) (b) (c)

Figure 2: (a) 90° hooks anchored into confined core, (b) U-bars used around outer vertical reinforcement (c) 135° hooks used in conjunction with a tie (NZS 3101:2006 2015).

2.2 End region crushing

In NZS3101:2006 (A2), confinement of the compression zone was required only if the depth of the neutral axis exceeded a critical limit, c_c, shown in Equation (1). Once the limit is exceeded the actual length to be confined, c', is determined as the neutral axis depth less 70% of the critical limit and as low as $0.5c_c$, as shown in Equation (2) below:

$$c_c = \frac{0.1\varphi_{ow}L_w}{\lambda} \quad (1)$$

$$c' \geq c - 0.7c_c \text{ but not less than } 0.5\ c_c \quad (2)$$

where c' = length of wall to be confined, c_c = critical neutral axis depth, φ_{ow} = ratio of base overstrength moment to earthquake induced moment, L_w = wall length, λ = factor based on the plastic hinge ductility class (limited ductile plastic region or ductile plastic region).

During the CERC investigation several examples of wall compression failures were partially attributed to the uncertainty of the axial loads. Interactions and deformation compatibility between structural elements resulted in significantly larger axial loads on RC walls than were assumed during design or assessment. To militate against the uncertainly in axial load, prevent compression failures, and increase ductility, it was recommended that the end region confinement extend over the entire compressive zone SESOC (2013). This recommendation was subsequently introduced into NZS3101:2006 (A3).

2.3 *Local bar buckling in wall web*

Following the 2011 Christchurch earthquake, it was observed that local buckling of vertical reinforcement in the central portion of the wall had occurred, even when the wall end regions appeared to be well confined (SESOC 2013). From this observation it is hypothesised in the SESOC (2013) interim design guidance that buckling initiated at the web portion of the wall as a result of cyclic yielding of longitudinal web bars in tension followed by load reversal, which put the bars into compression. To improve longitudinal web bar stability, a new clause has been inserted into NZS3101:2006 (A3) (Cl 11.4.5.3) as suggested by CERC and SESOC, which requires cross-ties to be provided in the web of walls designed with a limited ductile (LDPR) or ductile plastic hinge region (DPR) when limits on shear force, cover depth, bar spacing and curvature are exceeded. Two testing programmes have previously demonstrated the effect of web ties on response. Kuang and Ho (2009) and Hube et al. (2014) used ties in the web region on one wall specimen each and found displacement ductility increased by approximately 60% while the energy dissipation capacity increased by 100% and 30%, respectively. The improved performance can be fully attributed to the use of web cross-ties in these particular tests as no other variables were altered in both studies. There is scope to investigate the effect of web ties in a more comprehensive study that will cover a range of wall parameters.

2.4 **Global wall buckling**

Global buckling failure of structural concrete walls was noted in the 2011 Christchurch (Fig. 1c) and 2010 Chile earthquakes. In an effort to prevent this failure mode, the limiting clear height-to-thickness ratio has been reduced from 30 in NZS3101:2006 (A2) to 20 in NZS3101:2006 (A3) for axial load ratios, $P/A_gf'_c$, above 0.2. CERC (2012), Elwood (2013) and Wallace et al. (2012) have all suggested that the effect of wall height-to-thickness ratio on response should be carefully investigated accounting for other parameters such as expected loading and deformation. Wall thickness requirements for the stability of the wall boundary region in plastic hinges has remained unchanged in NZS3101:2006 (A3) – one of the few standards that includes this criteria. The minimum permitted reinforcement diameters in the plastic hinge region was also not changed.

Also related to global buckling, NZS3101:2006 (A3) imposes an axial load limit of $0.3A_gf'_c$ and comments that the influence of wall elongation on axial load should be considered. CERC (2012) provided no specific recommendations on this, instead suggesting that a procedure be developed to quantify the increase in axial load due to wall elongation. It is noted that global buckling failures have been observed in wall experiments with axial loads below $0.3A_gf'_c$ and across a range of height-to-thickness ratios (Oesterle et al. 1976; Paulay & Goodsir 1985).

2.5 **Minimum vertical reinforcement**

Lightly reinforced concrete walls exhibited particularly poor performance in the Canterbury earthquakes (Lu et al. 2015, Sritharan et al. 2014). Minimum steel provisions in NZS3101:2006 (A2) were intended to account for both thermal and shrinkage effects and to ensure that the yielding moment would exceed the cracking moment. Observed damage from the Canterbury earthquakes of lightly reinforced concrete walls designed to both standard versions indicated that many of these formed only a few primary cracks along the wall height with limited secondary crack formation. Consequently, longitudinal steel reinforcement yielding was highly localised at the crack locations, leading to premature bar fracture. The SESOC report has hypothesised one primary reason for the limited crack distribution to be the large actual-to-specified concrete strength (SESOC, 2013). SESOC suggests closer consultation with concrete suppliers to advise of the maximum (as well as the minimum) acceptable concrete strength suitable for the design. For walls with minimum reinforcement per NZS3101:2006 (A2), test results reported by Lu et al. (2015) have confirmed formation of localised cracks. To address this issue, the minimum vertical reinforcement requirement in the end region has been doubled in NZS3101:2006 (A3). A further study is currently under way to validate these changes (Lu et al. 2015).

2.6 Distributed reinforcement

SESOC puts an emphasis on the importance of designing walls with lumped reinforcement at the end regions, particularly for walls designed with LDPR or DPR. It is believed that having lumped reinforcement in the high tensile strain zone will encourage the formation of a distributed flexural cracking pattern rather than concentration of cracking at a few locations, leading to a larger plastic hinge length, which increases wall ductility (SESOC 2013). This recommendation needs validation, as lumping reinforcement to the wall ends and leaving only the minimum steel requirement in the web region, as suggested by SESOC, may not always be the most favourable solution. For example, Kuang & Ho (2008) have demonstrated that by moving reinforcement to the boundaries the web exhibited a more shear-dominated response making the wall prone to a brittle failure. To minimise this response Sritharan et al. (2014) concluded from previous test observations that at least 40% of the total longitudinal reinforcement should be placed within the web.

3 WALL DATABASE

To help assess the recent amendments to NZS3101:2006 (A2), a database was developed that parameterises experimental testing of rectangular RC walls with ductile detailing and/or exhibiting a ductile response. One of the primary objectives in developing the database was to map the database with current New Zealand design and construction practice. As current practice in New Zealand typically favours rectangular walls, this database did not include barbell, T-shaped, L-shaped or any other unconventional cross section shapes, as well as repaired or perforated specimens. The tests included in the database were all pseudo-static, predominantly with reversed-cyclic loading protocols. Relative to existing databases of structural wall tests (e.g. NEEShub Shear Wall Database (Lu et al. 2010)), this database puts significant emphasis on parameterising the end region of the wall, where most of the energy dissipation and nonlinear behaviour is expected to occur. This will allow for improved assessment of the relationship between end region detailing, ductility, and failure mode.

Not dissimilar to the other databases, this database includes parameterisation of wall geometry, loading conditions, material properties (nominal and tested), and reinforcement detailing (sizes and spacing) in the end region and the central web region. Results reported for each test include yield and ultimate drift, drift at key damage states (e.g. spalling, buckling), and maximum shear demands at the base. Yield drift was graphically estimated from the load-deformation plots provided in literature as 1.33 times the wall drift at 75% of maximum base shear. Ultimate drift was estimated at failure, which was considered to occur when the load at the peak of a cycle first dropped below 80% of the overall peak load and did not return to this level. A number of calculated parameters are included in the database, such as reinforcement ratios (longitudinal and transverse) in different regions (flange and web), shear span ratio, axial load ratio, wall slenderness and the analytically-determined neutral axis depth. Lastly, a number of these calculated parameters were normalised by NZS3101:2006 (A2) provisions as a way of assessing the code limits by comparison to experimental data.

Figures 3a-i provide histograms that categorise test parameters. The database includes over 140 test specimens from 30 test programmes conducted in Australasia, North and South America, and Europe over the last 30 years. Parameter definitions are as follows: aspect ratio = clear wall height normalised by wall length; shear span ratio = ratio of base moment to base shear normalised by wall length; slenderness ratio = wall height normalised by wall thickness; end region reinforcement ratio = area of longitudinal reinforcement to gross concrete area in the confined end region; and bar slenderness ratio = transverse reinforcement spacing normalised by longitudinal bar diameter in the end region. Failure modes are defined as follows: flexure crushing = crushing of the wall end region, shear-flexure = shear failure after yielding in flexure reduces shear strength; global buckling = out-of-plane buckling of all or part of the wall; flexure buckling = failure by local buckling of longitudinal bars and/or fracture; lap splice = failure of a lap splice connection; shear = failure by diagonal tension or shear sliding.

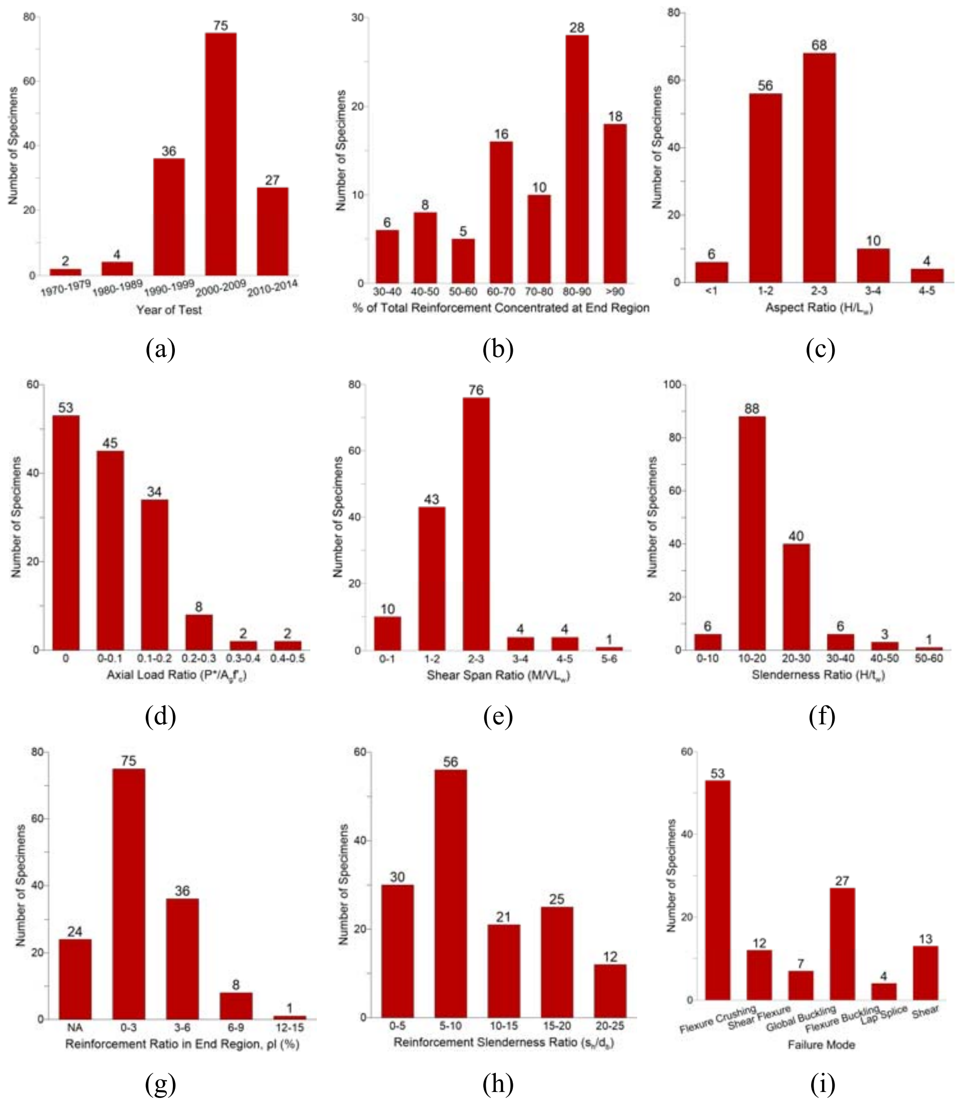


Figure 3: Summary of selected parameters from the database.

4 HYPOTHESIS TESTING USING THE DATABASE

An assessment of selected NZS3101:2006 (A2) recommendations summarized in Section 2 is provided using the database summarized in Section 3. Revisions are advised where deemed necessary.

4.1 *Confinement Depth*

To assess the influence of extending the depth of confinement into the central portion of the wall on the overall response, the confined boundary length, L_c, was normalised by the neutral axis depth, c, for walls in the database and plotted against the ultimate drift and displacement ductility (the ratio of ultimate to yield drift), μ_Δ, attained in the test. The results are presented in Figure 4 and Figure 5, respectively, noting that walls without confinement or those that were not tested to failure are not included. Global buckling failure was also excluded as this failure type is closer associated with the height-to-thickness ratio and axial load than confinement (Paulay & Priestley 1993). Despite significant scatter in Figure 4 data, there is a general trend indicating that a larger ratio of confined depth to neutral axis results in larger ultimate drifts. Figure 5 shows no apparent trend in ductility with confinement depth.

In Figure 4, data points to the right of the dashed line represent all tests where the confinement depth extended beyond the neutral axis depth, i.e. tests with fully-confined compression zones. Generally, these test specimens were able to achieve an ultimate drift of at least 1.5%. Although many tests with a confinement depth less than the neutral axis depth (those to the left of the dashed line), i.e. tests with partially confined compression zones, performed well, on the whole these tests show less sensitivity to confinement depth, which is evident by the large vertical spread of data points. This indicates that variability in other end region details, e.g. the transverse reinforcement ratio, is likely having a greater influence on performance.

Both plots in Figures 4 and Figure 5 indicate that all walls with L_c/c greater than one correspond to axial loads below $0.1A_gf'_c$. Conversely, walls with axial loads exceeding $0.1A_gf'_c$, noting that larger axial load corresponds to larger neutral axis depth, all have L_c/c ratios of less than one. Wall tests should therefore be conducted to investigate the effect of L_c/c ratios greater than one on walls with axial load exceeding $0.1A_gf'_c$.

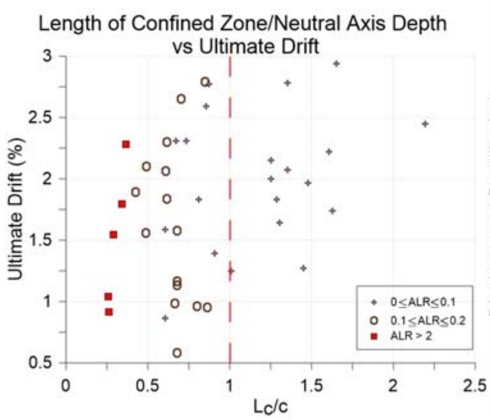


Figure 4: Effect of neutral axis depth confinement on drift capacity.

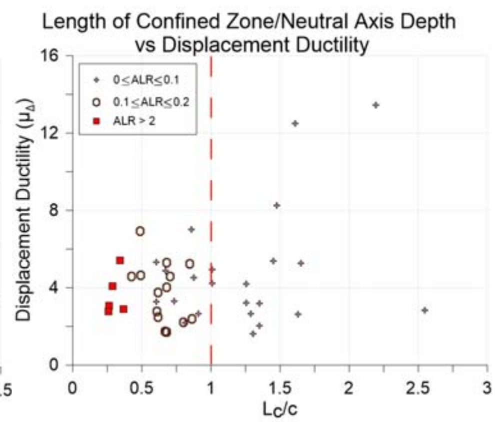


Figure 5: Effect of neutral axis depth confinement on displacement ductility.

4.2 *Concrete Core Crushing*

During design-level earthquakes, once spalling of cover concrete at the wall end region has occurred, the axial and flexural compressive stresses are resisted by the confined core. As well as extending confinement deeper into the wall, increasing the transverse reinforcement ratio in the end region (defined as the ratio of transverse reinforcement area to gross concrete area in the end region) can also improve concrete ductility in compression and delay the onset of crushing. In an attempt to quantify a limiting value on this parameter that will likely prevent crushing failure from occurring at moderate ductility demands, it is plotted against axial load ratio in Figure 6.

It is evident in Figure 6 that concrete crushing failure of specimens is prominent over a wide range of axial load ratios when the wall boundary transverse reinforcement ratio is below 0.4%. It was also found that 80% of these brittle crushing failures occurred before a displacement ductility for a ductile plastic region (DPR) of 3 is reached. Therefore, any ratio below 0.4% is insufficient in providing enough confinement to delay concrete core crushing for walls with limited ductile ($\mu_\Delta \geq 1.25$) or ductile plastic hinge regions ($\mu_\Delta \geq 3$). In the NZS3101:2006 (A3) revision, the required confinement reinforcement area, A_{sh}, for a wall designed for either level of ductility is determined as:

$$A_{sh} = \alpha s_h h'' \frac{A_g^*}{A_c^*} \frac{f'_c}{f_{yh}} \left(\frac{c}{L_w} - 0.07\right) \quad (3)$$

Where A_{sh} = total area of hoops required, α = factor dependent on ductility class, s_h = vertical hoop/tie spacing, h''= concrete core dimension measured perpendicular to the direction of hoops bars to outside of peripheral hoop, A^*_g = gross area of boundary, A^*_c = confined area of boundary, f'_c = specified compressive concrete strength, f_{yh} = yield strength of hoops, c = neutral axis depth and L_w = wall length. This equation has no minimum limit on the area of confinement to be used. Although most common construction geometries and detailing in NZ result in a wall boundary transverse reinforcement ratio above 0.4% using Equation (3), it is suggested that a minimum is still incorporated for walls with limited ductile or ductile plastic hinge regions ($\mu_\Delta \geq 3$).

4.3 *Local Bar Buckling*

Bar slenderness is defined as the ratio of the unsupported bar length to the bar diameter (Bae et al. 2005) and is a key parameter controlling longitudinal reinforcement susceptibility to buckling. At the wall boundary, the unsupported length of longitudinal reinforcement is equal to the spacing of transverse reinforcement, s_h. The relationship between the bar slenderness ratio (s_h/d_b) at the wall end region and the drift at which longitudinal reinforcement buckling first occurs is provided in Figure 7 for tests in the database. Tests were not included if the drift at which buckling was first observed was not reported.

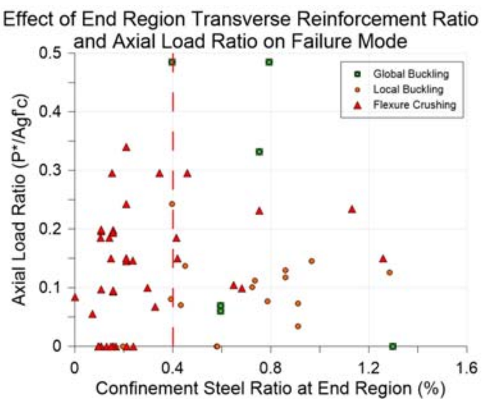


Figure 6: Effect of transverse reinforcement ratio and axial load on failure mode.

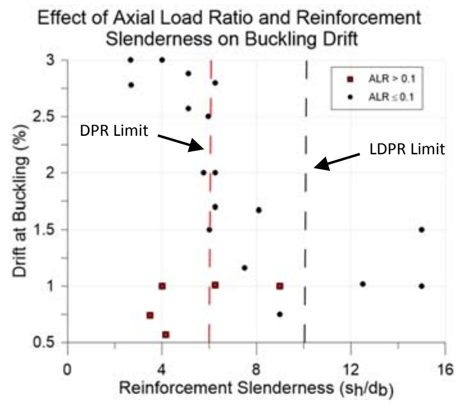


Figure 7: Effect of bar slenderness ratio on buckling drift.

An increase in bar slenderness corresponds to a lower drift at buckling (Fig. 7), as expected. This trend is evident for tests with with axial loads below $0.1A_gf'_c$. When axial load exceeds $0.1A_gf'_c$ it appears that buckling typically occurs at lower drifts, likely due to higher compression strains induced in the longitudinal reinforcement; sufficient data is not available to develop a conclusive trend. It is evident from the plot in Figure 7 that drift ratios in excess of 2.5% are achieved in all tests with $s_h/d_b < 5$ and $P/A_gf'_c \leq 0.1$. $s_h/d_b = 5$ is the critical buckling slenderness ratio determined in multiple studies on isolated reinforcement bar segments (Bae et al. 2005). The limit on s_h/d_b currently prescribed by NZS3101:2006 (A3) for walls with a DPR is 6. A slenderness ratio of 6 in Figure 7 corresponds to high performance variability (indicated by larger vertical scatter) and was therefore deemed inappropriate for design purposes. Comparatively, for walls with a LDPR the s_h/d_b limit is 10 and although in Figure 7 there is limited data at this reinforcement slenderness ratio, interpolating the general trend would indicate a significant reduction in buckling drift. This may be problematic as walls with a LDPR are expected to achieve a displacement ductility greater than 1.25. Therefore, it may be most appropriate to limit the s_h/d_b ratio to 5 in limited ductile and ductile plastic hinge regions. Further study is required to fully assess reinforcement slenderness recommendations.

5 CONCLUSIONS

Due to observations of unexpected failure modes in RC walls following the Christchurch earthquake, a number of revisions related to RC structural walls have been made to NZS3101:2006 (A2) following recommendations provided by SESOC (2013) and CERC (2012). These revisions were aimed at addressing key issues such as end region and web region confinement, anchorage detailing, and minimum reinforcement requirements. Most of these recommendations were based on professional judgement and available previous research. A database was created that parameterises tests performed on RC walls with rectangular cross sections. This database is unique from existing ones in that it provides detailed parameterisation of the end region of the wall, where failure often initiates in flexure-controlled walls. The database is instrumental in mapping the extent of the international testing space and in assessing the NZS3101:2006 (A3) code provisions to be published this year. Through initial assessment using the database, it was found that confining the full neutral axis as recommended by SESOC (2013) does not result in higher ductility levels but can provide improved drift capacity at low axial loads. A potentially effective alternative may be to provide ties throughout the central wall region, as this has been shown through limited previous testing to significantly improve wall performance. Additionally, it was found that a longitudinal reinforcement slenderness ratio (s_h/d_b) of 5 or less is appropriate in delaying buckling of longitudinal reinforcement to drifts greater than 2.5% for walls with an axial loads less than $0.1A_gf'_c$. Lastly, a minimum transverse reinforcement ratio of 0.4% is recommended in walls designed with limited ductile or ductile plastic hinge regions, as this limit was found to significantly reduce susceptibility to concrete crushing failure. Creation of the database is the first step in a multi-year study to improve seismic performance of ductile concrete walls. The above recommendations will be further investigated through a focused experimental programme.

6 REFERENCES

Bae, S., Mieses, A.M. & Bayrak, O., 2005. Inelastic Buckling of Reinforcing Bars. *Journal of Structural Engineering*, 131(2), pp.314–321.

CERC, 2012. *Section 1 : Summary and recommendations – Volumes 1-3*, Wellington.

Elwood, K.J., 2013. Performance of concrete buildings in the 22 February 2011 Christchurch earthquake and implications for Canadian codes. *Canadian Journal of Civil Engineering*, 40(8), pp.759–776.

Hube, M. a. et al., 2014. Seismic behavior of slender reinforced concrete walls. *Engineering Structures*, 80, pp.377–388.

Kuang, J.S. & Ho, Y.B., 2009. Seismic Behavior and Ductility of Squat Reinforced Concrete Shear Walls with Nonseismic Detailing. *ACI Structural Journal*, 2(105), pp.225–232.

Lu, Y. et al., 2015. Cyclic Testing of Reinforced Concrete Walls with Distributed Minimum Vertical. *Journal of Structural Engineering (submitted)*.

NZS 3101:2006, 2012. Concrete Structures Standard Part 1 - The Design of Concrete Structures (Amendment 2).

NZS 3101:2006, 2015. Concrete Structures Standard Part 1 - The Design of Concrete Structures (Amendment 3 public comment version).

Oesterle, R.G. et al., 1976. *Earthquake Resistant Structural Walls - Tests of Isolated Walls*, Skokie, Illinois.

Paulay, T. & Goodsir, W.J., 1985. The Ductility of Structural Walls. *Bulletin of the New Zealand National Society for Earthquake Engineering*, 18(3), pp.250–269.

Paulay, T. & Priestley, M.J.N., 1993. Stability of Ductile Structural Walls. *ACI Structural Journal*, 90(4), pp.385–392.

SESOC, 2013. *Interim Design Guidance: Design of Conventional Structural Systems Following the Canterbury Earthquakes*,

Sritharan, S. et al., 2014. Understanding Poor Seismic Performance of Concrete Walls and Design Implications. *Earthquake Spectra*, 30(1), pp.307–334.

Wallace, J.W. et al., 2012. Damage and implications for seismic design of RC structural wall buildings. *Earthquake Spectra*, 28(SUPPL.1), pp.281–299.

Lu, X. et al., 2010. Shear Wall Database, Network for Earthquake Engineering Simulation (database).

7 ACKNOWLEDGEMENTS

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