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Deformation limits for design and assessment of RC walls

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ABSTRACT

Comparing deformation demand to deformation capacity limits is a necessary step in the design and assessment of reinforced concrete (RC) walls to achieve the required level of ductility under earthquake loading. Results from testing of large-scale, well-detailed RC walls at the University of Auckland have been analysed in conjunction with a large database of previously tested RC walls in order to assess existing deformation limits in New Zealand design and assessment standards. It was found that the methodologies used to derive existing deformation demand and capacity limits are inconsistent and the limits poorly represent the trends in test data. A new mechanics-based model that uses material strain limits is proposed to capture the observed variation of deformation capacity, particularly with respect to the ratio of neutral axis depth to wall length (c/L_w) and the ratio of transverse reinforcement spacing to longitudinal bar diameter (s/d_b) at the wall end region. The new model is proposed for implementation into the NZ Seismic Assessment of Existing Buildings Guideline as it is shown to be more accurate and consistent in estimating the deformation capacity of RC walls compared to the existing models. It is also demonstrated that the proposed model can be used to derive deformation demand limits suitable for implementation into NZS 3101:2006; therefore achieving consistency between the deformation limits that are used for design of new buildings and assessment of existing ones.

1 INTRODUCTION

Deformation limits are critical in the design and assessment of RC walls because damage and strength degradation in the walls are deformation-controlled phenomena. Deformation *demand limits* are used to ensure that the maximum design deformation expected in the wall plastic hinge does not exceed the lower-bound deformation capacity as previously determined through experimental testing. Deformation demand limits are specified in terms of curvature ductility, K_d, in the New Zealand Concrete Structures Standard, NZS 3101:2006-A3 (NZS 3101:2006, 2017). K_d is calculated assuming all plastic deformation is lumped in a theoretical plastic hinge length, L_p. Deformation *capacity limits* are used in the assessment of existing RC wall buildings to verify that the RC wall can sustain the expected deformation demand in an earthquake event without loss of strength. Deformation capacity limits should provide a realistic (probable) estimate of deformation capacity; they should not be overly conservative (as this would lead to unnecessary retrofit) and they should not be optimistic of the expected wall performance (as this will provide false confidence in the structure's integrity). The NZ Seismic Assessment of Existing Buildings Guideline (NZSEE, 2017) (referred to as the 'NZ Assessment Guideline' hereafter) contains limits on material strains to estimate the deformation capacity in terms of plastic rotation, θ_p.

Deformation demand and capacity limits should satisfy several requirements to ensure structural safety, and should also be practical in their application. It is understood that safe structural designs must possess a deformation capacity that exceeds deformation demand by some factor of safety. It follows that deformation demand limits imposed by NZS 3101:2006-A3 should always exceed the deformation capacity limits used for assessment in the NZ Assessment Guideline. Until recently, the probable deformation capacity of RC walls in the NZ Assessment Guideline was either determined through material strain limits, or by referencing the demand limits in the NZS 3101:2006-A3; the latter clearly being a very conservative approach and not representative of probable capacity. Given the inherent relationship between deformation demand and deformation capacity, it is also desirable that the two sets of limits are founded upon the same theoretical basis and are correctly proportioned relative to each other. As explained in the following section, this is presently not the case. Finally, the limits should be balanced in simplicity for implementation into coded documents, and effectiveness in capturing variation in deformation capacity due to different wall characteristics.

This paper evaluates the deformation demand limits in NZS 3101:2006-A3 and the deformation capacity limits in the NZ Assessment Guideline with respect to their (i) relative magnitudes to one another and (ii) absolute accuracy with respect to a comprehensive database of RC wall tests. The experimental data are also used to derive a new deformation capacity model based on mechanics of materials. The proposed model is used to derive limits for both deformation demand and capacity; thus achieving consistency between the design and assessment of RC walls.

2 EXISTING DEFORMATION CAPACITY LIMITS IN NEW ZEALAND

The deformation capacity limits in NZS 3101:2006-A3 and the NZ Assessment Guideline are summarised in Table 1. The curvature ductility demand limits (K_d) in NZS 3101:2006-A3 are prescribed depending on the ductility classes to which the wall design belongs to, either: Nominally Ductile, Limited Ductile, and Ductile. The K_d limits for each class were derived in a study by Dhakal and Fenwick (2008) from a limited dataset consisting of only seven Ductile walls (four of which were coupled or in a dual system) and 20 singly reinforced walls classified as Nominally Ductile. The plastic rotation limits corresponding to the curvature ductility demand limits can be determined with assumptions for the plastic hinge length, yield rotation and yield curvature, also outlined in Table 1.

The NZ Assessment Guideline provides a method to determine maximum curvature capacity through a plane-section moment-curvature analysis with imposed strain limits on the steel and concrete materials. The concrete strain limit expression is taken from fib Bulletin 24 (fib, 2003) while the maximum strain for steel is given as 0.06 (this may be reduced further to account for the reinforcement susceptibility to buckling based on Alvarado et al., (2015)). The second approach of determining deformation capacity (termed the moment-rotation method) is not considered in this paper as it directly references the demand limits in NZS 3101:2006-A3.

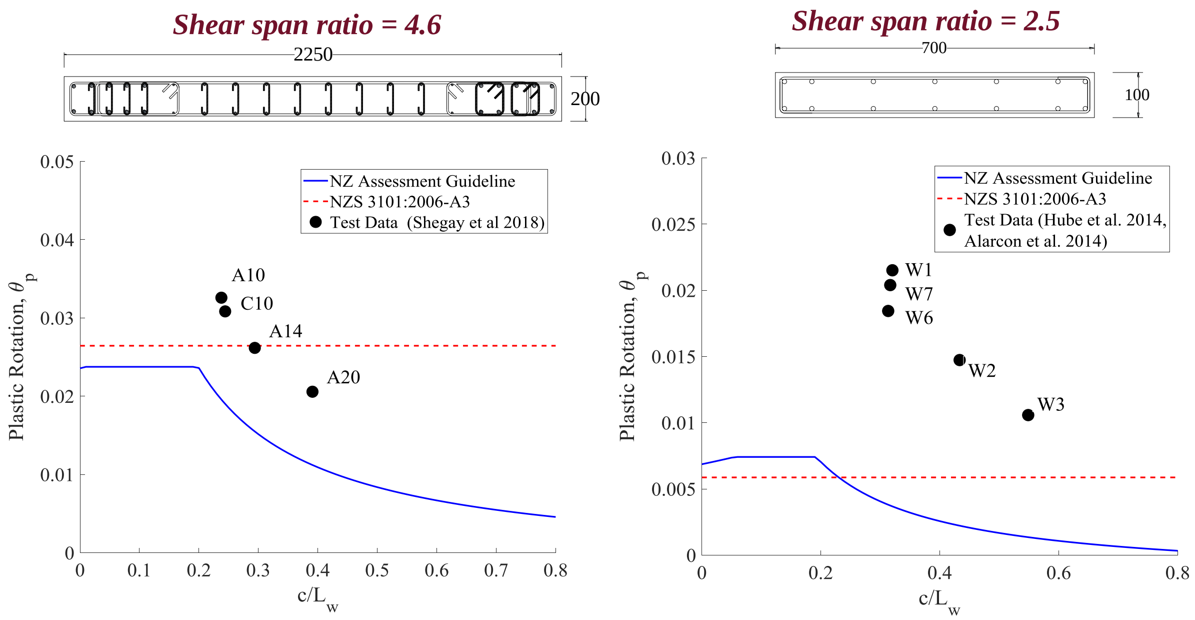
Table 1: Deformation limits in NZS 3101:2006-A3 and the NZ Assessment Guideline.

	Confined end region	**Unconfined end region**	**Notes**
NZS 3101:2006 – A3 – deformation demand limits			
Nominally Ductile Limited Ductile Ductile	$K_d = N/A$ $K_d = 9$ $K_d = 16$	$K_d = 4$ $K_d = N/A$ $K_d = N/A$	$\theta_p = K_d L_p \phi_y - \theta_y$ Where: $\phi_y = 2\varepsilon_y / L_w$ $L_p = min\begin{cases} 0.15\, M/V \\ 0.5 L_w \end{cases}$
NZ Assessment Guideline – deformation capacity limits			
Concrete strain limit, ε_{cm}	$min\begin{cases} 0.004 + \frac{0.6\rho_{st} f_{yh} \varepsilon_{su}}{f'_{cc}} \\ 0.05 \end{cases}$	0.004	$\theta_p = (\phi_{cap} - \phi_y) L_p$ $L_p = kh_{eff} + 0.1 L_w + L_{sp}$
Steel strain limit, ε_{sm}	$min\begin{cases} 0.6\varepsilon_{su} \\ 0.06 \end{cases}$	$min\begin{cases} 0.6\varepsilon_{su} \\ 0.06 \end{cases}$	
Curvature capacity, ϕ_{cap}	$min(\frac{\varepsilon_{cm}}{c}, \frac{\varepsilon_{sm}}{d-c})$	$min(\frac{\varepsilon_{cm}}{c}, \frac{\varepsilon_{sm}}{d-c})$	

K_d= Curvature ductility = $\phi_p/\phi_y + 1$, ϕ_p = plastic curvature capacity in plastic hinge, ϕ_y = yield curvature, ε_y= yield strain of the longitudinal reinforcement not exceeding 0.0021, θ_p=plastic rotation limit, θ_y=elastic rotation at top of plastic hinge (determined by integrating the linear yield curvature profile), L_p = estimated plastic hinge length, M/V = base moment to base shear quotient, L_w= wall length, ε_{cm}= concrete compressive strain limit, ε_{sm}= reinforcement tension strain limit, ε_{su}= strain at ultimate stress of the reinforcement, ρ_{st} = volumetric ratio of confinement reinforcement in the confined end region, f_{yh} = yield stress of the confinement reinforcement, f'_{cc} = confined compression strength of concrete, ϕ_{cap} = section curvature capacity, c = neutral axis length determined at ε_c=0.004, d = effective depth of longitudinal tension reinforcement, $k = 0.2(f_u/f_y - 1) \leq 0.08$, $L_{sp} = 0.022 d_b f_y$ Priestley et al. (2007), f_u= ultimate stress of the longitudinal tension reinforcement, f_y = yield stress of the longitudinal reinforcement, h_{eff} = effective height of the wall, d_b = diameter of the longitudinal reinforcement.

2.1 Comparison of relative limit magnitudes

To compare the deformation demand and capacity limits, both are plotted against the neutral axis depth to wall length ratio (c/L_w) in Figure 1a and Figure 1b for walls designed with Ductile and Nominally Ductile plastic hinge regions, respectively. As the deformation capacity limits in the NZ Assessment Guidelines are dependent on wall detailing, the comparisons in Figure 1 are made with respect previously tested walls with cross-sections that are 'typical' of the ductility class. The cross-sections are also shown in Figure 1. For Ductile walls, the NZS 3101:2006-A3 and NZ Assessment Guideline limits are similar at c/L_w ratios below 0.2. At higher c/L_w ratios, the deformation capacity limits in the NZ Assessment Guidelines reduce rapidly, while the NZS 3101:2006-A3 demand limits remain constant. As a result, walls with $c/L_w > 0.2$ can be designed to deformation demands higher than the assumed deformation capacity. A similar observation is made in Figure 1b for Nominally Ductile walls. The demand and capacity plastic rotation limits are similar at $c/L_w < 0.22$, and the demand limits exceed capacity limits at $c/L_w > 0.22$. As stated previously, deformation demand limits should be lower than the deformation capacity limits over the full range of demands because demand limits are based on lower-bound (conservative) estimates of deformation capacity, whereas capacity limits are intended to provide a probable estimate.



(a) With confined end regions (Ductile walls)

(b) Without confined end regions (Nominally Ductile walls)

Figure 1: Comparison of deformation limits for walls with and without confined end regions expressed in terms of plastic rotations using representative wall sections shown, (a) data from Shegay et al. (2018); (b) data from Alarcon et al. (2014) and Hube et al. (2014).

3 ACCURACY OF EXISTING LIMITS

3.1 Database

To compare the accuracy of the demand and capacity limits with respect to data from experimental wall tests, 71 walls were selected from a previously compiled wall database (Shegay et al., 2015). The ranges of key wall characteristics included in the database are summarised in Table 2. Experimental tests were excluded from the database if any one of the following criteria were met:

(i) failure of the wall load carrying capacity was primarily the result of longitudinal reinforcement splice failure, shear sliding/diagonal shear crack failure or out-of-plane global buckling failure

(ii) wall subjected to bi-directional lateral loading

(iii) wall shear span ratio $(\frac{M}{VL_w}) < 1.5$

(iv) asymmetric/monotonic loading protocol used in testing

(v) non-constant axial load applied during testing

(vi) longitudinal reinforcement ductility characteristics not compliant with either AS/NZS 4671 (2001) or ASTM A706/706M (2016) (hardening ratio, $\frac{f_u}{f_y}, \leq 1.15$).

The neutral axis depth was determined at a maximum concrete strain of ε_c=0.004 from a moment-curvature analysis (using a plane-section assumption, tested material properties, and material models of Hognestad (1951) and Menegotto and Pinto (1973) for steel and concrete, respectively). Displacement capacity, δ_u, was defined as the lateral displacement at which the measured lateral load resistance first dropped below 80% of the maximum measured lateral strength. For walls where the location of the reported lateral displacement does not correspond to the effective height (h_{eff}) of the wall (e.g., tests where moment was applied at the top of the wall), the lateral displacement at the effective height was extrapolated using an assumed cracked section bending stiffness of $0.35E_cI_g$ and shear stiffness of $0.4E_cA_g$ (where E_c is the Young's modulus of concrete, I_g is the gross cross sectional second moment of inertia and A_g is the gross cross sectional area of the wall). Non-rectangular walls are under-represented in the final database (total of 10 included), as limited relevant test data are available in literature. Ductile-classed walls were identified from the 71 walls as those that satisfy wall thickness (t_w) provisions of NZS 3101:2006-A3 ($t_w \geq 7t_{min}$, where t_{min} is the minimum wall thickness (as defined in Cl. 11.4.3.2)), and have a transverse reinforcement spacing to longitudinal bar diameter ratio (s/d_b) of less or equal to 6. Walls that did not satisfy these criteria by default belong to the Limited/Nominally Ductile class. It can be observed from the statistics in Table 2 that the selected walls span a wide range of demands (axial load ratio, $P/A_gf'_c$ = 0-50%; c/L_w = 0.02-0.7), geometries (M/VL_w = 1.5-6; L_w/t_w = 5-19) and reinforcement detailing (longitudinal reinforcement ratio, ρ_l = 0.8-12%; s/d_b = 1.8-17).

Table 2: Summary of the minimum, mean, maximum, and coefficient of variation (COV) of geometric, detailing, and demand parameters in the wall database.

Wall	$\frac{^aP}{A_gf'_c}$	$\frac{^bM}{VL_w}$	$\frac{^cV_u}{A_g\sqrt{f'_c}}$, **(MPa)**	$\frac{L_w}{t_w}$	$^d\rho_l$	$\frac{s}{d_b}$	$\frac{f_u}{f_y}$	$\frac{c}{L_w}$	$\frac{^e\delta_u}{h_{eff}}$, **%**	K_d
Min.	0	1.5	0.045	5	0.008	1.8	1.2	0.02	1.1	3.5
Mean	0.1	2.9	0.3	10.8	0.034	6.4	1.4	0.2	2.6	15.7
Max.	0.5	6.0	0.7	18.8	0.12	17	1.6	0.7	4.4	29.7
COV (%)	96	35	56	33	83	59	8	57	30	39

[a]Applied axial load ratio, [b]Shear span ratio, [c]Shear stress demand, [d]Reinforcement ratio in the boundary element, [e]Drift capacity = displacement capacity normalized by effective height.

To estimate the plastic rotation capacity of each wall in the database, all plastic deformation was assumed to be concentrated in the plastic hinge region at the base of the wall (lumped plasticity model) as shown in Figure 2. The plastic rotation was calculated using geometry from Equation (1) below. The corresponding curvature ductility capacity was calculated using the plastic hinge assumptions in Table 1.

$$\theta_p = \frac{\delta_u - \delta_y}{h_{eff} - (0.5L_p - L_{sp})} \qquad (1)$$

where $\delta_y = \frac{\phi_y h_{eff}^2}{3}$

where δ_y is the estimated wall displacement at yield using the estimated yield curvature, ϕ_y, from Table 1.

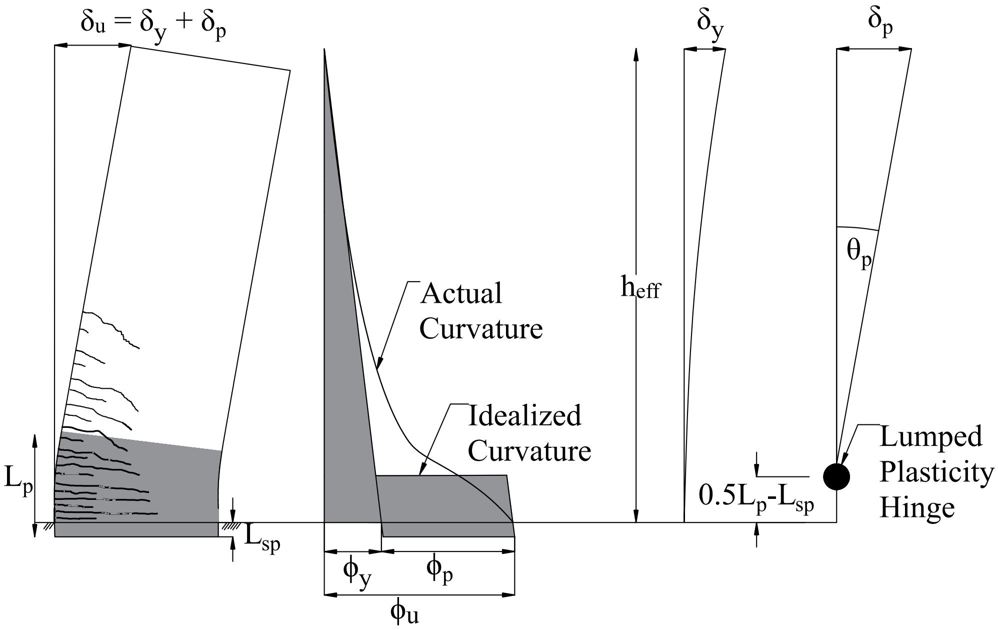
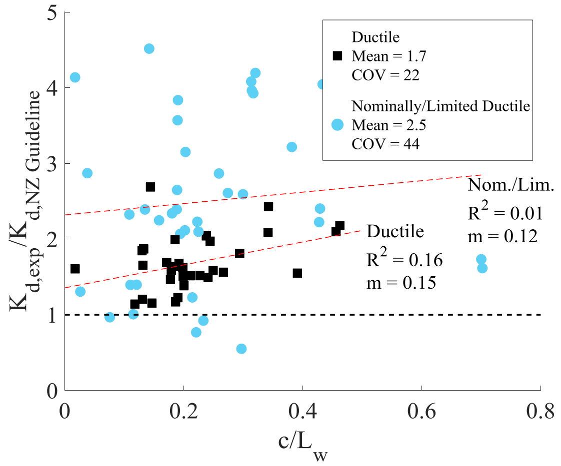


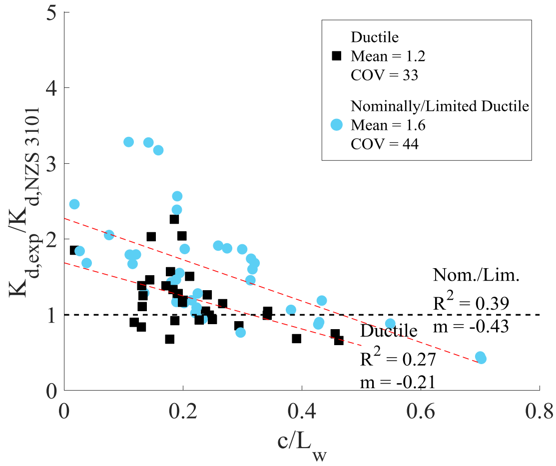
Figure 2: Lumped plasticity model based on Priestley et al. (2007).

3.2 Comparison of available limits to test data

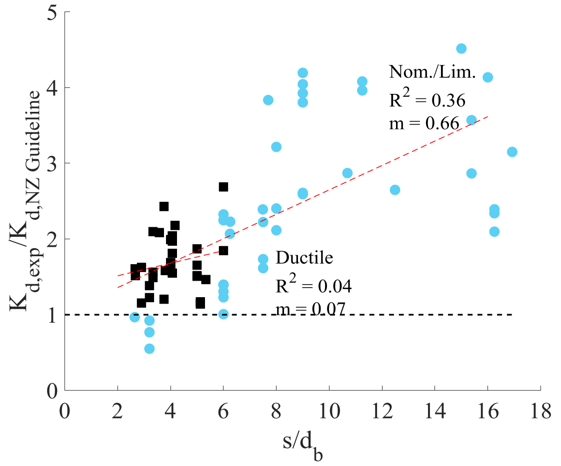
To compare the coded limits with curvature ductility determined from test data, the experimental K_d was normalized by the curvature ductility limits in NZS 3101:2006-A3 and the NZ Assessment Guideline. The normalized value was then plotted against c/L_w and s/d_b ratios in Figure 3a-b and Figure 3c-d, respectively. Together, these two parameters capture multiple key aspects of the wall, including longitudinal reinforcement ratio, shape of the cross-section, material properties, axial load ratio and effectiveness of restraint against longitudinal reinforcement buckling. In asymmetrical walls (T-shapes), the larger neutral axis, associated with the direction with lower drift capacity, was reported. The normalized values represent the 'accuracy' with which each coded limit reflects experimentally determined curvature ductility capacity. Ratios above and below unity in Figure 3 indicate that the code/guideline underestimates and overestimates the actual curvature ductility achieved in the experiment, respectively. The average ratio across all walls is reported in the plot legend. For assessment guidelines (where probable capacity is sought), normalized values of unity are expected while for design standards (where a lower bound capacity is desired) normalized values above unity are expected. Trend lines are fitted to each plot, and the coefficient of determination (R^2) and the trend line slope (m) are reported as indicators of the dependency of the limit accuracy on the x-axis parameter.



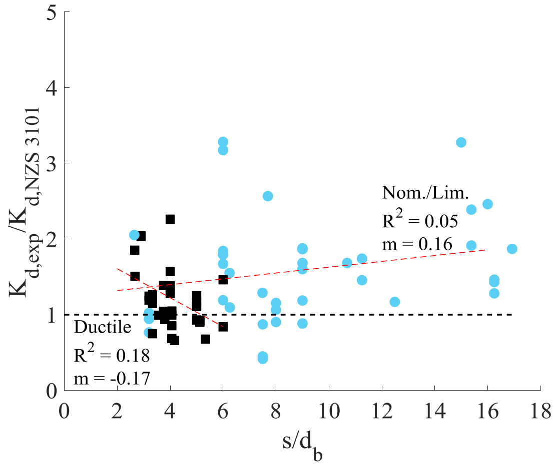
(a) NZ Assessment Guideline limits over c/L_w range



(b) NZS 3101:2006-A3 limits over c/L_w range



(c) NZ Assessment Guideline limits over s/d_b range



(d) NZS 3101:2006-A3 limits over s/d_b range

Figure 3: Curvature ductility limits for test walls relative to limits from NZ Assessment Guideline and NZS 3101:2006-A3 across a range of c/L_w and s/d_b ratios.

The accuracy of the NZS 3101:2006-A3 and NZ Assessment Guideline limits show dependency on the c/L_w ratio as indicated by high m and R^2 values (e.g. m = 0.21-0.43, R^2 = 0.27-0.39 in Figure 3b) and on s/d_b ratio (e.g. m = 0.17-0.66, R^2 = 0.18-0.36 in Figure 3c and 3d). These observations indicate that the deformation demand limits and the probable capacity calculation are not adequately considering the effect of c/L_w and s/d_b on the wall deformation capacity. The average prediction accuracy of the NZS 3101:2006-A3 demand limits is 1.2 and 1.6 for Ductile and Limited/Ductile walls, respectively; however, 20 of the 71 walls (28%) have a deformation capacity lower than the NZS 3101:2006-A3 limits. This is a concerning result as it suggests that 28% of walls may be designed for deformation demands that exceed the wall deformation capacity. The average prediction accuracy of the NZ Assessment Guideline is higher: 1.7 and 2.5 for Ductile and Limited/Ductile walls, respectively. Only four walls have an under-estimated deformation capacity using the NZ Assessment Guideline. Thus, the probable deformation capacity of the RC walls is generally under-predicted using the NZ Assessment Guideline. Both sets of limits also show considerable scatter which suggests poor consistency in the prediction accuracy.

3.3 Proposed Model

It was demonstrated in Figure 1 that the available deformation demand limits in the NZS 3101:2006-A3 standard and deformation capacity limits in the NZ Assessment Guideline are incorrectly proportioned because the permitted design demand exceeds the probable capacity. A comparison of the limits to experimental data in Figure 3 showed that the limits do not properly account for the variation in deformation capacity with changes in c/L_w and s/d_b. It was also apparent in Figure 3 that the demand limits are not conservative with respect to experimental data, while the probable capacity limits are over-conservative. In this section a model is proposed to account for the variation observed in Figure 3 and achieve correct proportioning between deformation demand and capacity limits.

From a plane section strain distribution, the curvature capacity can be calculated using Equation (2). Taking the yield curvature definition from Table 1, curvature ductility can subsequently be derived as a function of the extreme fibre compressive strain limit (ε_{cm}), the longitudinal reinforcement yield strain (ε_y), and c/L_w, as shown in Equation (3). It was found that the concrete strain limits of ε_{cm}=0.018 and ε_{cm}=0.012 produce reasonable fits to the experimental K_d data for Ductile and Limited/Nominally Ductile walls, respectively. The K_d limit using these strain values is plotted in black with the experimental K_d data in Figure 4a and Figure 4b. It was also determined that lower strain values of ε_{cm}=0.014 and ε_{cm}=0.008 can be used to establish a lower-bound envelope to the Ductile and Limited/Nominally Ductile wall data in Figure 4, respectively, that can be used as deformation demand limits in design.

$$\phi_{cap} = \frac{\varepsilon_{cm}}{c} \tag{2}$$

$$K_d = \frac{\varepsilon_{cm}}{2\varepsilon_y \left(\frac{c}{L_w}\right)} \tag{3}$$

It can be seen from Equation (3) that K_d will approach infinity at low c/L_w ratios. This is clearly not the case in real walls and is the outcome of assuming that deformation capacity is exclusively governed by compression strain. In reality, a tension-related failure mechanism will occur first when compression demands (represented by c/L_w) are low. Reinforcement buckling is a common precursor to a tension-governed wall failure. If compression demands are low, an RC wall can typically sustain additional deformation demand after buckling initiation; however, for simplicity, the point of buckling initiation will be considered to correspond to maximum curvature ductility capacity in this study. Typical buckling models use the total strain exertion in the reinforcement, ε_{sc} (defined as the sum of maximum concrete strain and reinforcement strain demands) to predict initiation of buckling. In this study, the relationship developed by Moyer and Kowalsky (2003) shown in Equation (4) is selected to determine ε_{sc}. This relationship was selected as it is based fundamentally on Euler buckling theory. Expectedly, the relationship in Equation (4) predicts higher strains with decreasing s/d_b ratio. As ε_{sc} is the total strain exertion the curvature capacity and curvature ductility capacity associated with ε_{sc} can be determined using Equations (5) and (6), respectively. By comparing to the experimental data, it was determined that $s/d_b > 5$ is too conservative (likely due to the assumption that initiation of reinforcement buckling results in wall failure), thus a minimum of K_{d_max} = 12 (corresponding to s/d_b = 5) is recommended for both Ductile and Limited/Nominally Ductile walls.

$$\varepsilon_{sc} = 3\left(\frac{s}{d_b}\right)^{-2.5} \tag{4}$$

$$\phi_{cap} = \frac{\varepsilon_{sc}}{L_w} \tag{5}$$

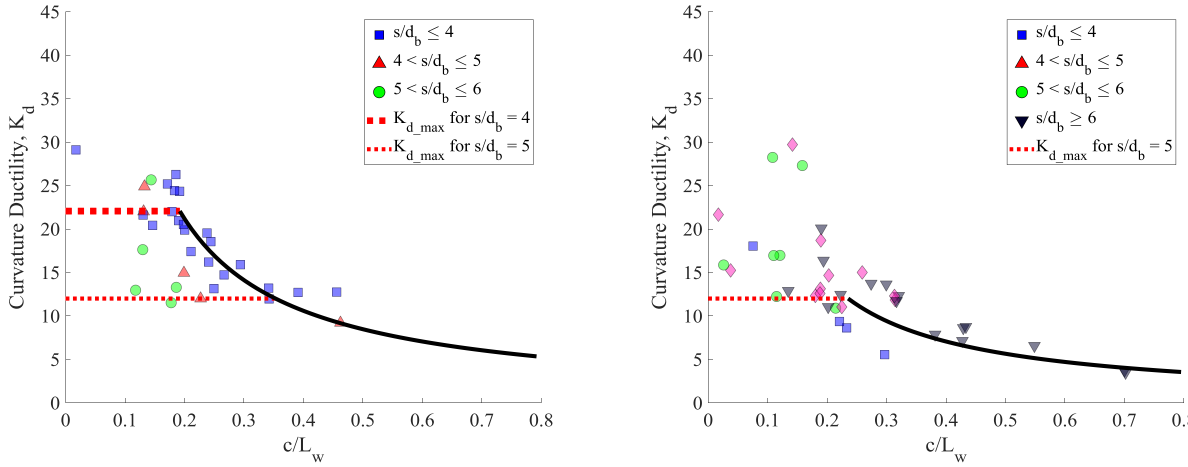
$$K_{d\_max} = \frac{\varepsilon_{sc}}{2\varepsilon_y} \tag{6}$$

By combining Equations (3) defining compression failure with Equation (6) defining reinforcement buckling criteria, the final proposed model for design and assessment of RC walls is recommended in Equation (7). The compression strain limits and K_{d_max} values for each wall class and limit type are summarised in Table 3. The final probable capacity limits for Ductile and Limited/Nominally Ductile walls are plotted with experimental data in Figure 4a and Figure 4b respectively. It can be seen that in both figures, the proposed limits reasonably approximate the K_d determined in experiments.

$$K_d = \frac{\varepsilon_{cm}}{2\varepsilon_y\left(\frac{c}{L_w}\right)} \leq K_{d\_max} \tag{7}$$

Table 3: Concrete compressive strain limits and K_{d_max} limits for the proposed model in Equation (7).

	Ductile	**Nominally/Limited Ductile**
Concrete strain limit for assessment (probable)	$\varepsilon_{cm} = 0.018$	$\varepsilon_{cm} = 0.012$
Concrete strain limit for design (lower bound)	$\varepsilon_{cm} = 0.014$	$\varepsilon_{cm} = 0.008$
K_{d_max}	12 for $s/d_b \geq 5$ 22 for $s/d_b \leq 4$ Linear interpolation for $4 \leq s/d_b \leq 5$	12



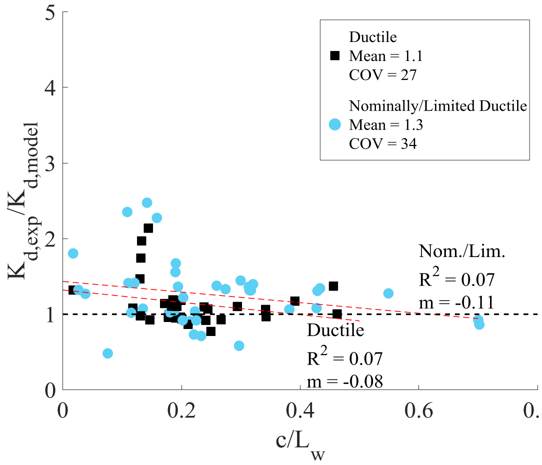
(a) Ductile walls　　**(b)** Limited/Nominally Ductile walls

Figure 4: Proposed deformation capacity limits plotted with curvature ductility capacity from experimental test data.

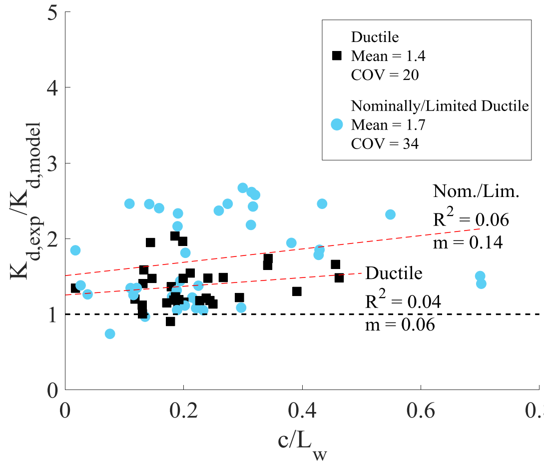
4 MODEL ACCURACY

Similarly to Figure 3, K_d values from experimental test data were normalized by curvature ductility capacity determined using the proposed model in Equation (7) and Table 3 and the results plotted against c/L_w and s/d_b in Figure 5. The following key improvements are evident:

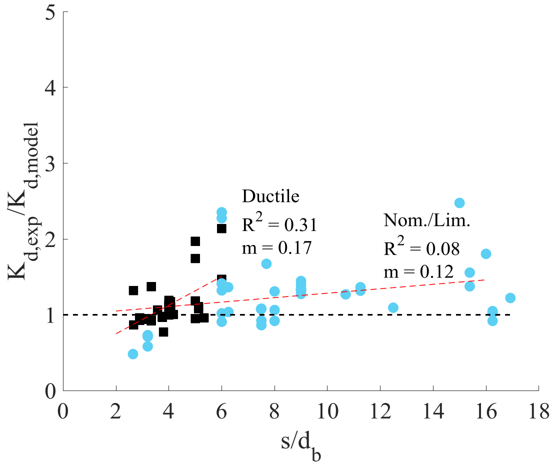
- The trend line gradients (m) and R^2 values using the proposed deformation capacity model are smaller for Ductile and Limited/Nominally Ductile walls than previously shown in Figure 3 using the NZ Assessment Guideline model. This is true with respect to c/L_w in Figure 5a (m = 0.08-0.11; R^2 = 0.07) and s/d_b in Figure 5c (m = 0.12; R^2 = 0.08). The exception to this is the apparent dependency of prediction accuracy of Ductile walls on s/d_b (m = 0.17; R^2 = 0.31); however, it is evident from Figure 5c that the trend is influenced by three largest values. These results indicate that the accuracy of the predicted probable deformation capacity on c/L_w and s/d_b has been significantly reduced using the proposed model compared to using the NZ Assessment Guideline model.
- Similarly, the proposed model for the NZS 3101:2006-A3 demand limits also show reductions of dependency to c/L_w in Figure 5b (m = 0.06-0.14; R^2=0.04-0.06) and s/d_b in Figure 5d (m = 0.04-0.17; R^2 = 0.02-0.09).
- The accuracy of the deformation capacity limits is much closer to unity as indicated by the mean accuracy of 1.1 and 1.3 (c.f. 1.7-2.5 in Figure 3) for Ductile and Limited/Nominally Ductile walls, respectively. A small level of conservatism in the probable deformation capacity estimate such as this is expected in wall assessment procedures.
- The mean prediction accuracy of the proposed design demand limits in Figure 5b/d are 1.4 and 1.7 for Ductile and Limited/Nominally Ductile walls, respectively. The conservatism indicated by these values confirm that the deformation demand limits are representative of lower-bound deformation capacity values, which is the desired outcome in design procedures.



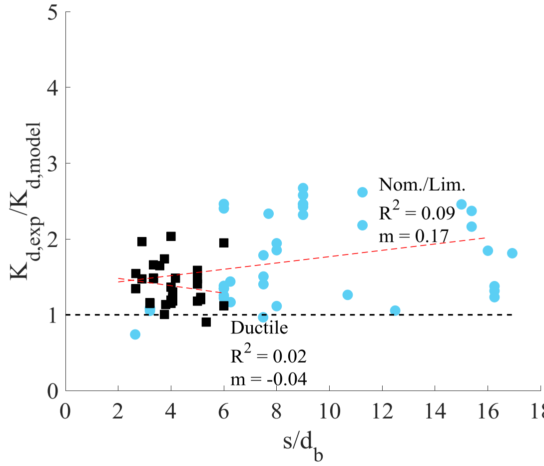
(a) Proposed model for probable capacity NZ Assessment Guideline c/L_w



(b) Proposed model for demand limits in NZS 3101 c/L_w



(c) Proposed model for probable capacity NZ Assessment Guideline s/d_b



(d) Proposed model for demand limits in NZS 3101 s/d_b

Figure 5: Measured curvature ductility capacity normalized by that estimated using the proposed model over a range of c/L_w and s/d_b ratios.

5 CONCLUSIONS AND RECOMMENDATIONS

Deformation demand limits in NZS 3101:2006-A3 and deformation capacity limits in the NZ Assessment Guidelines were investigated in this study. The demand and capacity limits were compared by their relative magnitudes in providing the expected proportioning for lower bound and probable deformation capacity, respectively. The limits were also evaluated in absolute terms by comparing to deformation capacities achieved in a database of RC experimental data. The following conclusions and recommendations are made:

(i) The deformation demand limits in NZS 3101:2006-A3 were found to be higher than the deformation capacity limits in NZ Assessment Guidelines. This finding is not consistent with expectation as it indicates that the maximum deformation demand to which RC walls are permitted to be designed is in excess of the wall probable deformation capacity.

(ii) In several cases, the deformation demand limits in NZS 3101:2006-A3 were found to be higher than the deformation capacity observed in experimental test data, suggesting that new walls maybe under designed. The accuracy of the deformation demand limits was also found be correlated to s/d_b and c/L_w ratios, which shows that the influence of these parameters on wall performance is not properly considered by the demand limits.

(iii) The NZ Assessment Guidelines deformation capacity limits were found to be highly conservative with respect to experimental data. The accuracy of the probable capacity limits was also found to be correlated to c/L_w and s/d_b ratios, suggesting the probable capacity limits do not properly consider the influence of these two parameters on wall performance.

(iv) A deformation capacity model based on material mechanics was developed and calibrated to experimental data. It was determined that reinforced concrete strain limits of $\varepsilon_{cm} = 0.018$ and $\varepsilon_{cm} = 0.014$ are suitable to define the probable (assessment) and lower bound (design) curvature ductility capacity of Ductile walls governed by a compression failure. For Limited/Nominally Ductile walls, the corresponding concrete strain limits of $\varepsilon_{cm} = 0.012$ and $\varepsilon_{cm} = 0.008$ are recommended to establish deformation capacity and demand limits, respectively.

(v) To account for walls failing due to a tension-controlled mechanism, the total strain exertion limit was introduced based fundamentally on Euler buckling mechanics. The strain exertion is a function of s/d_b ratio, which accounts reduced susceptibility of buckling with reduced transverse reinforcement spacing. Based on the experimental data, it was determined that in using the model, the s/d_b ratio need not be taken more than 5, which corresponds to a curvature ductility capacity of 12.

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