Capacity Design of Nominally Ductile Shear Wall Systems in New Zealand

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ABSTRACT

The current provisions of NZS 3101:2006 (and its interaction with NZS 1170.5:2004) provide ambiguous instruction on the requirements for the capacity design of nominally ductile shear wall systems. The recent release of the National Seismic Hazard Model update, along with Earthquake Design for Uncertainty document, has been a timely reminder of the criticality of capacity design, even for nominally ductile structures. This paper highlights the perceived confusing aspects of the current requirements of NZS 3101:2006 and proposes a design pathway for a variety of wall-based structural systems that addresses compliance and best-practice requirements.

INTRODUCTION

In regions of lower seismicity within New Zealand, concrete structures are often kept economic by being designed as nominally ductile, thereby avoiding the additional detailing requirements of NZS 3101:2006 (SNZ) for ductile structures. However, NZS 1170.5:2004 (SNZ) and NZS 3101:2006 still have resiliency (i.e. capacity design) requirements for nominally ductile structures that must be complied with. The pathway for this compliance, particularly for nominally ductile wall-based structures, is ambiguous.

It is the author's opinion that certain aspects of the capacity design process for nominally ductile wall-based structures may often by overlooked. Previously, there may have been a certain acceptance of this due to the perceived lower risk of such structures. However, the recent release of the National Seismic Hazard Model (Gerstenberger et al. 2022) update and the Earthquake Design for Uncertainty advisory (NZSEE, SESOC, NZGS 2022), has brought the criticality of capacity design to the fore.

This paper summarises the current capacity design requirements for nominally ductile wall-based structural systems and provides a proposed methodology for satisfying these requirements where ambiguity is perceived to be present.

NZS 3101 NOMINALLY DUCTILE DESIGN REQUIREMENTS

NZS 1170.5:2004 clause 5.6.1 requires Limited Ductile and Ductile structures to follow capacity design principles, while capacity design of Nominally Ductile structures is deferred to the appropriate material standard – NZS 3101:2006 in the case of concrete shear wall systems.

Clause 2.6 of NZS 3101:2006 provides the capacity design requirements of nominally ductile walls. Relevant excerpts are paraphrased below.

Clause 2.6.1.2 classifies structures into one of four ductility categories; Brittle, Nominally Ductile, Limited Ductile, and Ductile. A structure designed/proportioned so that it does not experience ductile demand at the ultimate limit state (i.e. μ = 1.0) is still considered a nominally ductile structure in accordance with the Standard (provided the design meets all the detailing requirements of the Standard).

Clause 2.6.1.3.1 categorises potential plastic regions, for the purposes of defining detailing requirements, into one of three categories; nominally ductile plastic regions (NDPR), limited ductile plastic regions (LDPR), and ductile plastic regions (DPR).

Clause 2.6.1.3.2 defines the material strain limits for each type of potential plastic region. The material strain limit is dependent on the level of detailing adopted for the potential plastic region. Nominally ductile systems may have plastic hinges with material strains that exceed the limits for NDPR's. The calculation of material strains is given within the clause.

Clause 2.6.1.3.4 defines the maximum allowable curvature ductility demands for plastic hinges, K_d, for NDPR's (provided in Table 2.4).

Clause 2.6.6.1 outlines the capacity design requirements for Nominally Ductile structures:

(a) They are proportioned such that only mechanisms allowable by clause 2.6.7 could develop when subjected to larger-than-anticipated seismic demands.
(b) When a mechanism could form which is not permitted for limited ductile or ductile systems, potential plastic regions shall be identified and detailed in accordance with the additional seismic design requirements of the appropriate sections of the NZS 3101:2006.

THE PROBLEM: MATERIAL STRAIN LIMITS

By setting ULS material strain limits in yielding elements to an acceptable level, we are ensuring that there is sufficient reserve strain capacity of avoid premature collapse at higher intensities of earthquake loading. It is the author's opinion that flexural strain limits in nominally ductile shear walls are rarely checked in practice. This may be due to lack of understanding of the requirement to do so, or a lack of understanding in how to do so. Additionally, no indication of a requirement to diagonally reinforce coupling beams is provided in the nominally ductile sections of the beam and wall design provisions.

THE PROBLEM: STRENGTH HEIRARCHY

Establishing a strength hierarchy is critical to ensuring potential plastic deformations occur in areas that are detailed to sustain them. This is addressed by NZS 3101:2006 clause 2.6.6.1 for nominally ductile structures. Subsection (a) refers the designer to clause 2.6.7, which is for <u>frame buildings</u>; no reference is made to sections relevant to wall buildings. Subsection (b) requires the designer to determine which mechanism(s) could develop and, if not in accordance with the allowable mechanism (i.e., for walls, hinging at the base), requires all potential plastic hinge regions to be identified and detailed in accordance with the additional seismic design requirements. This would be counter to the goal of keeping designs economic.

It is assumed that clause 2.6.6.1(a) should also refer to clause 2.6.8 for wall buildings. If so, clause 2.6.8.1 requires walls to fail in flexural hinging. Clause 2.6.8.2 requires the shear demand in all locations to consider flexural overstrength and dynamic amplification. However, no guidance is given on how to appropriately consider flexural overstrength and dynamic amplification for nominally ductile wall-based systems (as there is in clause C2.6.6.1(a) for frame systems).

The commentary to clause 2.6.1.3.4(e) acknowledges that for squat walls, "there is an interaction between flexure and shear and structural actions need to be considered together". It is therefore not immediately clear how designers can satisfy clause 2.6.8 for squat walls.

THE PROBLEM: MAXIMUM ACTIONS

The discussion around, and implementation of, capacity design often focuses on buildings in more seismically active regions, where ductility is more likely to be utilised to reduce ULS structural demands. In regions of lower seismicity, the need to utilise ductility to reduce ULS structural demands is often lower. Or other design requirements may necessitate structural proportioning that results in a structure that does not experience ductility demand at the ultimate limit state. The overstrength demands on such structures may require detailing that is onerous to the point of the design being challenged for 'overdesign'.

To mitigate this issue in steel structures, NZS 3404 has clause 12.3.3.4 which allows secondary members of category 3 (i.e. nominally ductile) structural systems to have an upper-bound design action derived from elastic (μ = 1.0, S_p = 0.9) demands.

NZS 3101:2006 has no 'maximum actions' section that is a parallel to that of NZS 3404. The only similar clause is 11.4.6.2 allowing the shear demand on limited ductile/ductile walls to be capped at nominally ductile demands. The inference of this clause is that shear failure is sufficiently ductile to sustain a ULS ductility demand of 1.25 when walls are detailed with the stricter requirements of clause 11.4. This goes against the convention of shear failure being considered as 'brittle'.

PROPOSED SOLUTION: CURVATURE CHECK – SINGLY REINFORCED WALLS

A simple plot to check the curvature limit of flexurally dominated, singly reinforced walls is derived below.

The deflection at the top of an elastic, flexure-dominated wall can be expressed as a function of the curvature at its base:

$\Delta_n = \frac{1}{3}\varphi_{base}{H_n}^2$ (Priestley, Calvi and Kowalsky, 2007)

The yield curvature for a shear wall is given by NZS 3101:2006 clause 2.6.1.3.2, with h taken as the length of the wall, L_w:

$$\varphi_y = \frac{2f_y}{E_s L_w} \; (f_y \leq 425MPa)$$

The curvature ductility limit of for singly reinforced walls is 0.8 (NZS 3101:2006 table 2.4):

$$\varphi_{lim} = \frac{1.6 f_y}{E_s L_w} \left(f_y \leq 425MPa\right)$$

The limiting deflection at the top of a cantilever wall can therefore be expressed as:

$$\Delta_{n,lim} = \frac{1}{3}\frac{1.6 f_y}{E_s L_w} {H_n}^2 = \frac{8 f_y {H_n}^2}{15 E_s L_w} \left(f_y \leq 425MPa\right)$$

By dividing both sides of the equation by the wall height, a drift limit for flexurally dominated singly reinforced walls as a function of the wall aspect ratio (H_n/L_w) can be derived (Figure 1):

$$\frac{\Delta_{n,lim}}{H_n} = \frac{8 f_y}{15 E_s}\frac{H_n}{L_w} \left(f_y \leq 425MPa\right)$$

If the drift limit is exceeded, then either more stiffness is required in the system, or the wall needs to be doubly reinforced. It is acknowledged that this limit ignores shear deformations and therefore errs on the side of conservatism.

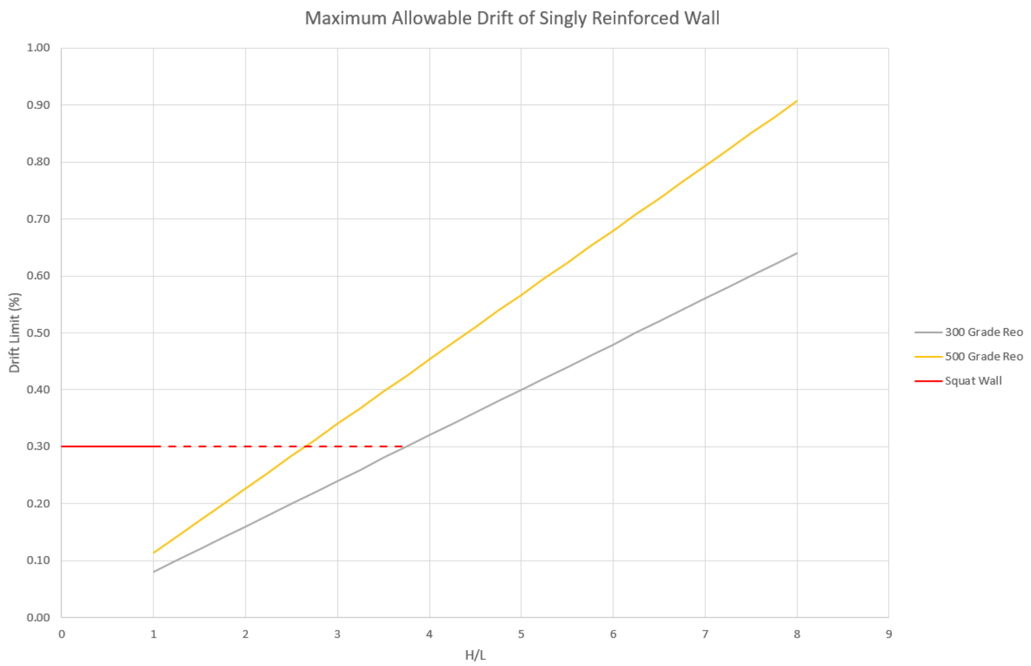

Figure 1: Maximum allowable drifts for singly reinforced walls.

PROPOSED SOLUTION: CURVATURE CHECK – DOUBLY REINFORCED WALLS

A simple plot to check the curvature ductility demand for flexurally dominated, doubly reinforced walls is presented below.

Paulay and Priestley (1992) derive the expression relating curvature ductility to displacement ductility for a cantilever wall (equation 3.59 of the text):

$$K_d = \mu_\varphi = 1 + \frac{\mu_\Delta - 1}{3(l_p/H_n)(1 - 0.5 l_p/H_n)}$$

For a nominally ductile system:

$$K_d = \mu_\varphi = 1 + \frac{0.25}{3(l_p/H_n)(1-0.5l_p/H_n)}$$

Multiplying the terms in the denominator by $(1/L_w)/(1/L_w)$:

$$K_d = \mu_\varphi = 1 + \frac{0.25}{3\left(\left(\frac{l_p}{L_w}\right)/\left(\frac{H_n}{L_w}\right)\right)\left(1 - 0.5\left(\frac{l_p}{L_w}\right)/\left(\frac{H_n}{L_w}\right)\right)}$$

The plastic hinge length for a wall is defined by NZS 3101:2006 clause 2.6.1.3.3:

$$l_p = 0.15\frac{M_e}{V_e} \leq 0.5L_w$$

Assuming a ratio of M_e/V_e of $0.7H_n$, and dividing by the wall length, the plastic hinge length can be expressed as a function of the wall aspect ratio:

$$\frac{l_p}{L_w} = 0.105\frac{H_n}{L_w} \leq 0.5$$

Substituting into the expression for K_d:

$$K_d = \mu_\varphi = 1 + \frac{0.25}{3\left(\left(0.105\frac{H_n}{L_w}, 0.5\right)_{min}/\left(\frac{H_n}{L_w}\right)\right)\left(1 - 0.5\left(0.105\frac{H_n}{L_w}, 0.5\right)_{min}/\left(\frac{H_n}{L_w}\right)\right)}$$

$$K_d = \mu_\varphi = 1 + \frac{0.25}{3\left(\left(0.105, \frac{0.5}{H_n/L_w}\right)_{min}\right)\left(1 - 0.5\left(0.105, \frac{0.5}{H_n/L_w}\right)_{min}\right)}$$

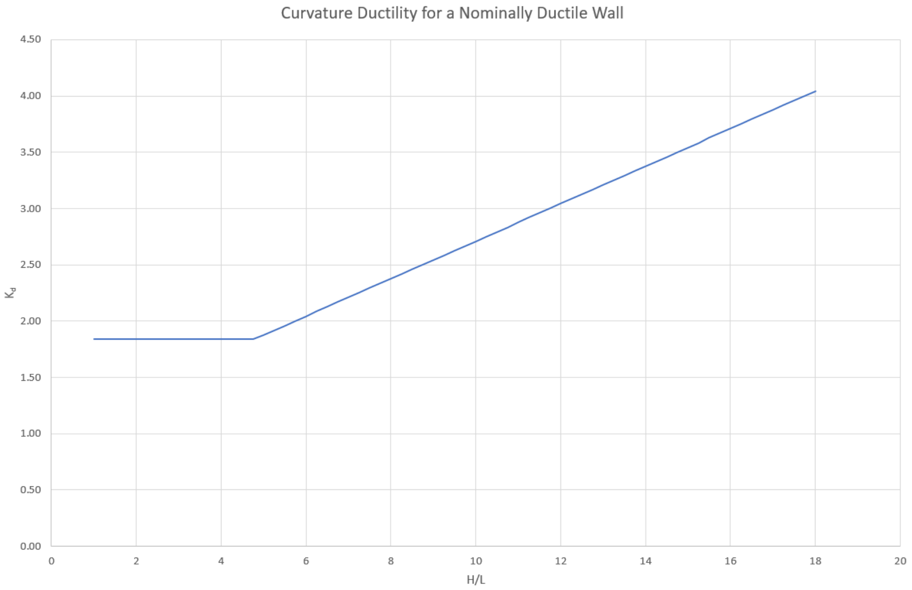

Figure 2: Curvature ductility demand as a function of aspect ratio for nominally ductile cantilever walls.

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Plotting this relationship (Figure 2) shows that the curvature ductility demand exceeds the limit for nominally ductile walls (4.0) once the aspect ratio is above 18. Beyond this, confinement of the end/boundary zones and/or limited ductile detailing would be required. As an aspect ratio of 18 is beyond of most typical designs, it indicates nominally ductile detailing is sufficient to comply with curvature ductility limits for most typical situations.

PROPOSED SOLUTION: SQUAT SHEAR WALLS

NZS 3101:2006 defines squat walls as those with an aspect ratio (H/L) equal to or less than 1.0. As the shear and flexural behaviour of such walls are coupled, the concept of separate moment and shear capacity design envelopes becomes irrelevant. Instead, the behaviour of squat walls is best represented via drift limits. Clause 2.6.1.3.4(e)(i) provides a drift limit of 0.3% for nominally ductile squat walls. The commentary for this clause references testing completed by Paulay, Priestley, and Synge (1982). Figure 3 shows the description of one of their test specimens and the corresponding hysteresis result. A 0.3% drift limit for the specimen would correspond to a displacement of 0.3% x 1500mm = 4.5mm. Observation of the hysteresis graph shows that the wall, which is singly reinforced with a confined end zone (i.e., a detailing typology towards the lower end of robustness), clearly has sufficient capacity to perform beyond this drift. It is therefore proposed that no other capacity design checks are required for nominally ductile squat walls provided the drift limit of 0.3%, and the detailing requirements of the Standard, are adhered to.

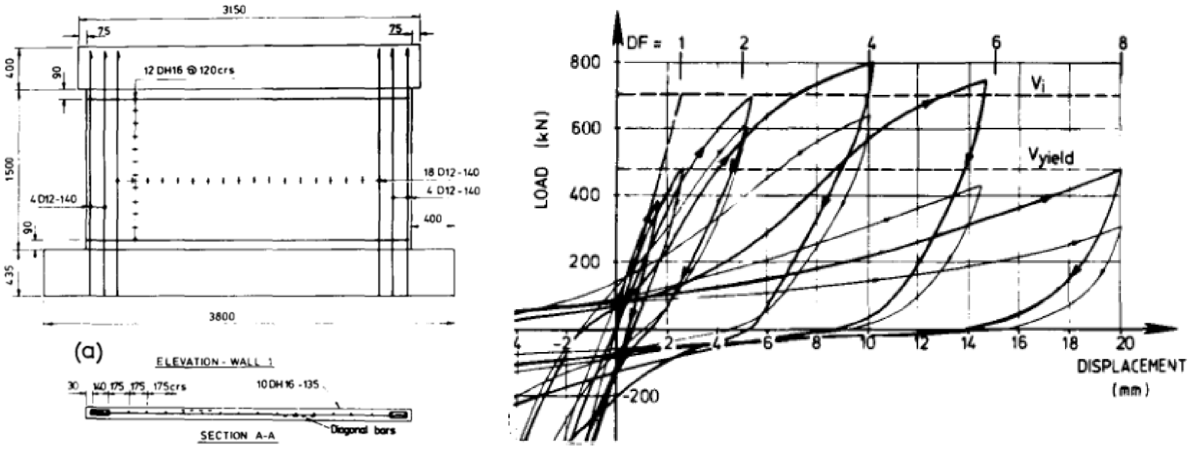


Figure 3: Squat shearwall test specimen and results, from Paulay, Priestley, and Synge (1982).

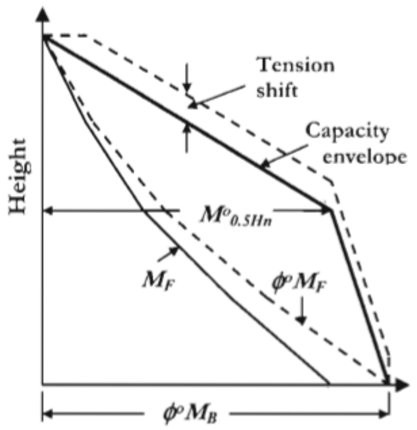
PROPOSED SOLUTION: STRENGTH HEIRARCHY AND MAXIMUM ACTIONS

NZS 3101:2006 clause 2.6.8.1 requires walls to fail in flexural hinging (ideally at the base of the wall). Clause 2.6.8.2 requires the shear demand in all locations to consider flexural overstrength and dynamic amplification. A proposed process to satisfy these requirements for nominally ductile cantilever shear walls follows.

Moment Demand Envelope

Wall base moments come directly from the seismic analysis. Reinforcing above the base of the wall must be sized to ensure hinging is prevented from occurring elsewhere. Priestley, Calvi and Kowalsky (2007) recognise that the moment envelope in a wall system is dependent on the intensity of the earthquake, i.e., the moment envelope is dependent on the ductility demand. They developed an expression for the moment envelope that is a function of the ductility demand (Figure 4). They note that “minor inelastic flexure action at levels above the base are acceptable” and therefore recommend that “flexural reinforcement areas at levels above the base be determined… without inclusion of a flexural strength reduction factor”.

Given that the overstrength factor in NZS 3101:2006 is 1.35 (clause 2.6.5.5(a)), the second half of the $C_{1,T}$ expression will always be negative for nominally ductile or elastic systems. Hence $C_{1,T}$ will be 0.4. For simplicity, assuming designers will use a strength reduction factor in their calculations, a 0.85 multiplier will be applied to the overstrength factor to give 1.35 x 0.85 = 1.15. This is in line with the overstrength factor adopted in the commentary to NZS 3101:2006 clause 2.6.6.1(a) for frame buildings. Therefore, the mid-height demand moment will be 0.4 x 1.15 = 0.46 x the nominal moment design capacity, simplified to 0.50 x the design moment capacity. I.e. a linear design moment capacity envelope from the base to the top of the wall can be used, see Figure 5 (a tension shift of $L_w/2$, as recommended by Priestley, Calvi and Kowalsky, is still included).



(a) Moment Capacity Envelope

$$M^o_{0.5H_n} = C_{1,T}\phi^0 M_{n,base}$$

$$C_{1,T} = 0.4 + 0.075T_i\left(\frac{\mu}{\phi^0} - 1\right) \geq 0.4$$

Figure 4: Priestley, Calvi and Kowalsky (2007) design moment envelope for shear walls.

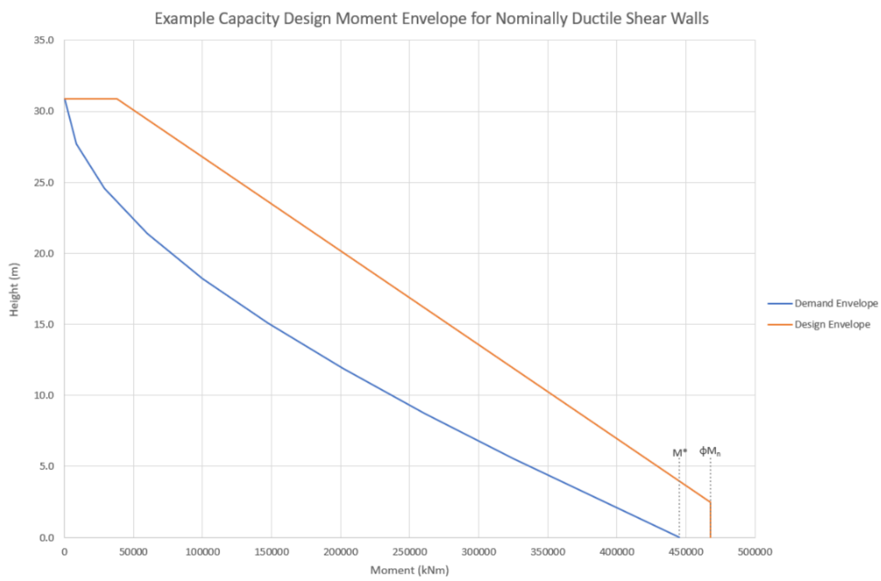


Figure 5: Example of proposed moment design envelope for a nominally ductile cantilever wall.

Shear Demand Envelope

Priestley, Calvi and Kowalsky have also developed an expression for a shear force envelope that is ductility dependent (Figure 6). The overstrength factor becomes:

$$\phi^0 = 1.35\phi M_{n,base}/M^*_{base}$$

If $\phi M_{n,base}$ is significantly larger than M* (e.g., due to minimum reinforcing requirements), then the overstrength shear demand at the base can become unrealistically large. An upper bound overstrength base shear of 1.5x nominally ductile shear demands is proposed, as this parallels the collapse limit load margin required by 1170.5. It is the author's opinion that research should be conducted to better specify a value for this upper bound limit. The calculation for the overstrength base shear demand in the wall then becomes:

$$V^o_{base} = \phi^0 \omega_v V^*_{base} \le 1.5V^*_{base,\mu=1.25}$$

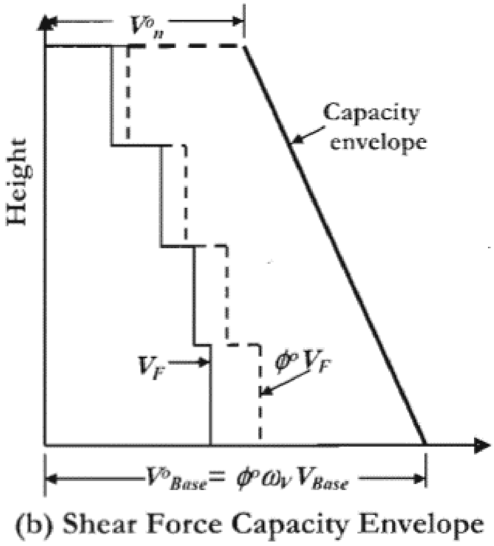
$$V^o_{base} = \phi^0 \left(1 + \frac{\mu}{\phi^0} C_{2,T}\right) V^*_{base} \le 1.5V^*_{base,\mu=1.25}$$

$$V^o_{base} = \left(\phi^0 + \mu C_{2,T}\right) V^*_{base} \le 1.5V^*_{base,\mu=1.25}$$

$$V^o_{base} = \left(1.35\phi M_{n,base}/M^*_{base} + \mu C_{2,T}\right) V^*_{base} \le 1.5V^*_{base,\mu=1.25}$$

The equations for $C_{2,T}$ and the overstrength shear demand at the top of the wall are as per Figure 6.

As a full capacity design procedure to NZS 3101:2006 clause 2.6.5 has not been followed, it is the author's opinion that the strength reduction factor should still be included in the calculation of the shear strength to resist the nominal overstrength shear demand.



$$V^o_{base} = \phi^0 \omega_v V^*_{base}$$

$$\omega_v = 1 + \frac{\mu}{\phi^0} C_{2,T}$$

$$C_{2,T} = 0.067 + 0.4(T_i - 0.5) \le 1.15$$

$$V^o_n = C_3 V^o_{base}$$

$$C_3 = 0.9 - 0.3T_i \ge 0.3$$

Figure 6: Priestley, Calvi and Kowalsky (2007) design moment envelope for shear walls.

COUPLING BEAM DETAILING AND MATERIAL STRAIN LIMITS

One of the major design decisions for coupled shear walls is whether or not the coupling beams require diagonal reinforcing. If practicable, it is desired to avoid diagonal reinforcing to save cost and time on site. As guidance in NZS 3101:2006 is minimal regarding diagonal reinforcing for nominally ductile systems, a literature review of well-known texts within New Zealand on the matter is presented below, followed by a proposed design process.

Literature Review; Coupling Beams and Diagonal Reinforcement

Paulay and Priestley (1992) note that:

- "In considering the behaviour of coupling beams it should be appreciated that during an earthquake significantly larger inelastic excursions can occur in such beams than in the walls that are coupled."
- "Many coupling beams have been designed as conventional flexural members with stirrups and some shear resistance allocated to the concrete. Such beams inevitably fail in diagonal tension… Unless the shear force associated with flexural overstrength of the beam at the wall faces can transmitted by vertical stirrups only, a diagonal tension failure will result."
- "When conventional shear reinforcement is based on capacity design principles, some limited ductility can be achieved."
- They recommend that diagonal reinforcing is used when the shear stress at beam ends exceeds $0.1(L/h)f'_c{}^{0.5}$.

Priestley, Calvi and Kowalsky (2007) highlight that "coupling beams would generally yield at a much lower level of response than would the walls". To illustrate this, they provided an example calculation that shows the yield drift for yielding at the base of the walls is circa 8.5x the yield drift of the coupling beams.

NZS 3101:2006 has several requirements for limited ductile and ductile coupling beams, presented in clause 11.4.9:

- Overstrength of coupling beams must consider the significant increase in axial loading on the beam due to restraint from the floor.
- Diagonal reinforcing shall not be used when coupling beam L/H > 4.
- Diagonal reinforcing must be provided when the overstrength shear stress exceeds that allowed for DPR's in beams ($0.16f'_c{}^{0.5}$ or $0.85f'_c{}^{0.5}$).
- Conventionally reinforced beams can be used when the shear stress is less than or equal to $0.3f'_c{}^{0.5}$.
- Conventionally reinforced beams must assume the shear strength provided by the concrete is zero, and that the axial force is zero for shear strength calculations.

Proposed Solution: Conventional Reinforcing Detailing

The rotation, and therefore curvature, at the ends of coupling beams can be calculated from the structural geometry and the wall drift (Figure 7):

$$\theta_{CB} = \theta_w(1 + (0.5L_{W1} + 0.5L_{W2})/L_{CB})$$

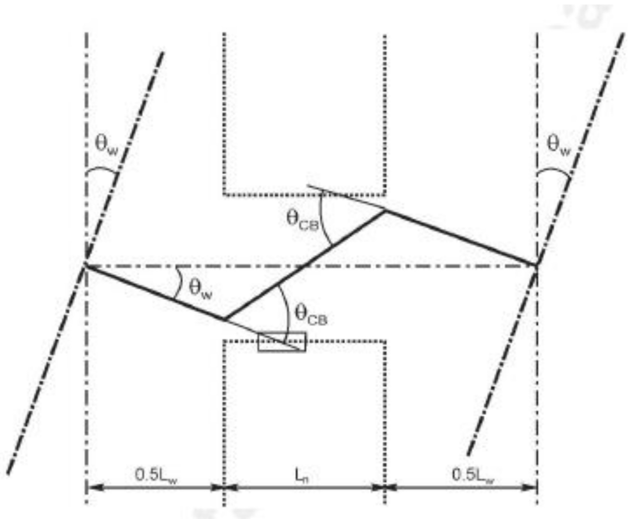


Figure 7: Definition of average shear strain rotation in coupling beams (NZS 3101:2006 figure C2.1A).

The plastic hinge length for a beam is given by (NZS 3101:2006 clause 2.6.1.3.3(a)):

$$l_p = 0.25k_p\, M_e/V_e \leq 0.5k_p h_{CB}$$

Where $k_p = (h_{CB}/d - 0.25) \geq 1.0$ (will always be 1.0 for a coupling beam)

$$l_p = 0.25\, M_e/V_e \leq 0.5h_{CB}$$

$$l_p = 0.25\, M_e/(2M_e/L_{CB}) \leq 0.5h_{CB}$$ (shear determined from overstrength moment)

$$l_p = L_{CB}/8 \leq 0.5h_{CB}$$

The maximum allowable rotation of the coupling beam is therefore:

$$\theta_{CB,max} = l_p K_d \varphi_y$$

$$\theta_{CB,max} = (L_{CB}/8\,, 0.5h_{CB})_{min} K_d \frac{f_y}{E_s h_{CB}}\ (f_y \leq 425MPa)$$

A relatively quick, preliminary calculation with realistic wall and beam geometries can show that it becomes difficult to find a scenario where $\theta_{CB} < \theta_{CB,max}$ for wall drifts above 0.5% when K_d is limited to 3 (the limit for nominally ductile beams, from NZS 3101:2006 table 2.4). I.e., the idea of a ‘nominally ductile coupling beam’ quickly becomes unfeasible.

The literature review noted that the concrete contribution to the shear strength should be ignored for coupling beams. This is already part way to complying with the detailing requirements of NZS 3101:2006 clause 11.4.9.5, and therefore part way to allowing K_d to be taken as 11 (limited ductile beam) or 19 (ductile beam). On this basis, a minimum of limited ductile detailing is recommended for conventionally reinforced coupling beams in nominally ductile systems. This enables K_d to be taken as 11 and will likely result in curvature limits not being a governing factor in the need for diagonal reinforcement. Instead, shear stress limits are likely to govern.

CONCLUSIONS AND RECOMMENDATIONS

The wording of NZS 3101:2006 is ambiguous in how a correct capacity design procedure for nominally ductile shear wall systems in New Zealand can be followed. The following solutions have been proposed to enable designers to design nominally ductile shear wall systems to comply with the Standard:

- Squat shear walls do not need to follow a capacity design procedure provided the required 0.3% drift limit (and all other detailing requirements) is maintained.
- A drift-based plot for quickly checking curvature limits on nominally ductile, cantilever, singly reinforced walls is presented.
- An aspect ratio-based plot for quickly checking curvature limits on nominally ductile, cantilever, doubly reinforced walls is presented.
- A linear capacity design moment demand envelope, from the design strength at the base, to zero at the top (based on Priestley, Calvi and Kowalsky (2007)).
- A capacity design shear force demand envelope following Priestley, Calvi and Kowalsky (2007) with a nominal overstrength factor of 1.15.
- An upper bound shear demand of 1.5x nominally ductile demands.
- Coupling beams always be detailed to a minimum of limited ductile detailing, which should result in the need for diagonal reinforcing be determined by shear stresses only.

It is recommended that next update to NZS 3101 considers:

- Clarifying the required moment and shear capacity design envelopes for nominally ductile shear walls, as it already does for nominally ductile moment frames. This paper offers a solution for consideration.
- Establishing research into determining an appropriate ‘maximum actions’ limit for shear demands in shear walls.

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