Performance of Limited Ductility Reinforced Concrete Walls In Low to Moderate Seismicity Regions

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ABSTRACT: Reinforced concrete walls provide cost-effective solutions for building structures to resist earthquake or wind induced lateral loads, therefore walls are frequently incorporated as primary load resisting system in buildings. Owing to their in-plane stiffness, structural walls can minimise inter-storey drifts and safeguard deteriorated structures from collapse. In regions of low to moderate seismicity, walls are sized and detailed based on gravity and wind loads without explicit checks on seismic performance and thereby in many cases they are considered vulnerable to brittle shear failure. The inherent complexity and intertwined mechanics governing shear response render backbone behaviour of shear-critical lightly reinforced wall panels not fully understood nor properly encapsulated within the framework of design practice. This paper investigates the damage progression of lightly reinforced walls in buildings with vertical irregularity (also known as set-backs). In this context, shear strength degradation model has been proposed and validated against representative tested walls in the literature.

1 INTRODUCTION

In regions of low to moderate seismicity, such as Australia, the vast majority of the building stock consists of RC buildings where the primary lateral load resisting system comprises limited ductile RC walls. In such regions, the design paradigm for walls is often restricted to gravity and wind considerations, with little attention directed towards seismic performance. The direct implication of this falls in place when analysing performance of limited ductile walls in irregular and complex geometric settings, namely; set-back and transfer structures.

Research on transfer structures; structures featuring discontinuity in load path and geometry across the height, highlighted complex interaction between shear performance of walls and structural setting (Su et al., 2002, Su and Cheng, 2009). Shear-concentrations ensued within the vicinity of transfer system were associated with out-of-plane deformation of transfer plates/girders and higher mode effects. Other research on irregular structures highlighted disproportionate seismic shear demand across the height of structures (Lee and Ko, 2007), however, most of studies were focused on the susceptibility of forming soft stories, without examining the adverse effect of shear-strained walls.

In this paper, a numerical case study example is presented for the purpose of addressing some shortcomings in the conventional procedures employed for assessing shear demand and deformation on structural walls. The latter stems from the lack of rigorous requirements stipulated in design codes, specifically in the low to moderate seismicity regions. Local shear mechanism is further illustrated by means of a nonlinear numerical model, the latter emphasises on the crucial need for performance-based framework to ensure resilient and safe design. Moreover, a preliminary shear capacity envelope for lightly loaded and reinforced RC walls is proposed and validated against representative walls in the literature.

2 VULNERABILITY ASSESSMENT: CASE STUDY

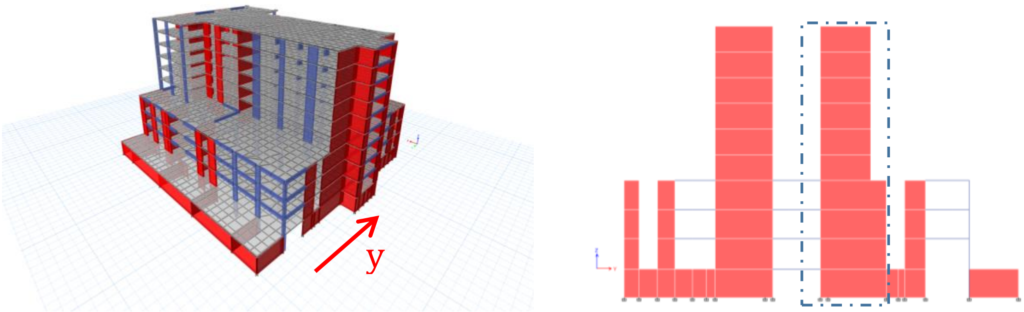
In the following subsections, the results from linear and nonlinear analysis of the case study structure are presented and examined.

2.1 Linear static and dynamic analysis

The case study structure is a medium rise ten-storey building designed and constructed in a low seismicity region. The structure features a vertical irregularity above the 3rd floor, where a geometric setback (in plan area) is imposed up to the roof level. This structural configuration was intentionally selected since higher mode effects can be considered to not govern dynamic behaviour and further, walls in low to medium rise structures are typically subjected to mild axial pre-compression forces.

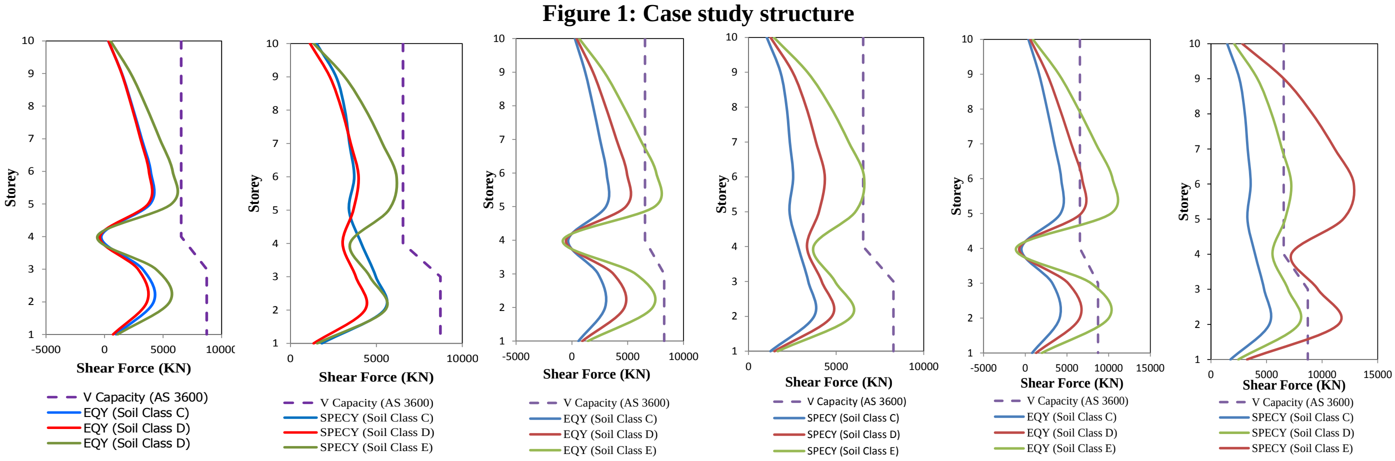
For the purpose of demonstrating wall performance, a case study edge wall was selected (figure 1-b). The examined wall spans all floors, with variable cross sections (8400x350 below podium and 7500x300 for all tower levels) and reinforcement ratios; up to podium level: 0.21% (V) and 0.4% (H), the reinforcement is then curtailed to 0.17% (V) and 0.22% (H). Results from dynamic and equivalent static analysis (based on AS 1170.4 recommendations) along the global y-direction for the examined wall for different return periods and subsoil classes are shown in figure 2. Soil Classes: C, D and E, are nominated in the figures in accordance to AS 1170.4 recommendations.

As demonstrated in the figure 2, design elastic shear forces, in many instances, exceeded computed capacity (based on AS 3600 recommendations) under different seismic load intensities and/or subsoil classes. Shear distribution features two distinct peaks across the height of the structure: one at podium level and one in the tower portion. There are many factors that contribute towards such shear distribution. In-plane eccentricity resulting from the offset between mass and rigidity centres (figure 3-b) induced substantial torsional moments especially in the lower portions of the structure. The latter can also be inferred from the displacement profiles given in figure 3-a, whereby significant floor displacements along the x-direction were prompted. Additionally, although higher mode effects were presumably negligible, the results of modal analysis has shown substantial mass participation at higher modes (90% cumulative mass participation ratio summed up to mode 10 and 11 along the x and y direction respectively). These torsional modes aggravated shear concentration (spikes) specifically at the edge and corner walls.



(a) 3D rendering of FE model **(b) Examined wall pier**

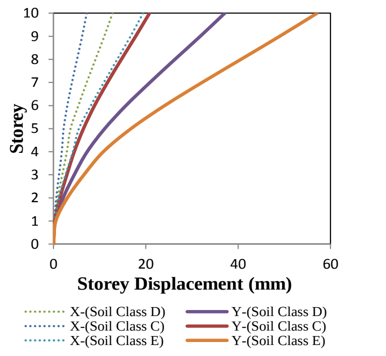
Figure 1: Case study structure

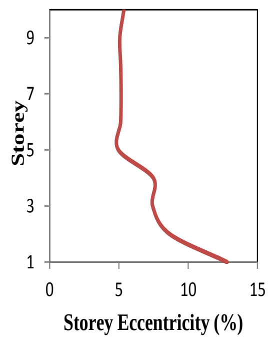


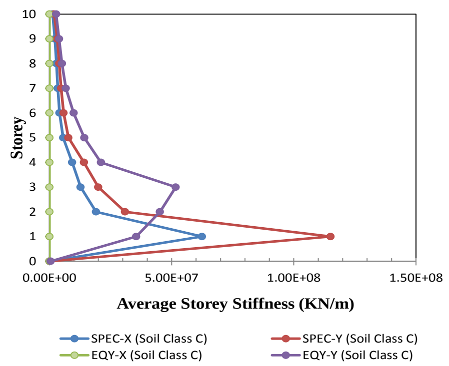
(a) 1/500 yrp (AS 1170.4) **(b) 1/1500 yrp (AS 1170.4)** **(c) 1/2500 yrp (AS 1170.4)**

Figure 2: Linear analysis results (SPECY; RSA analysis along y /EQY; Equivalent horizontal load along y)

So far, the discussion pertained to shear strength limit in the specific wall piers (figure 1-a), to better conceptualise performance of the structure in general and the examined wall pier in specific, the subsequent discussion is capitalised within the framework of deformations. With reference to figures 3-c/d, it can be shown that the abrupt stiffness gradient along the height of the structure, and specifically, at the podium-tower interface, has resulted in high inter-storey drifts, which in elastic analysis, directly translate to higher shear gradient between stories. Although subtle information was obtained through linear elastic analysis, the consequence for such shear concentration at higher levels is not explicitly demonstrated. To this end, a detailed nonlinear quasi-static (pushover) analysis was performed and the results are summarised in the following subsection.



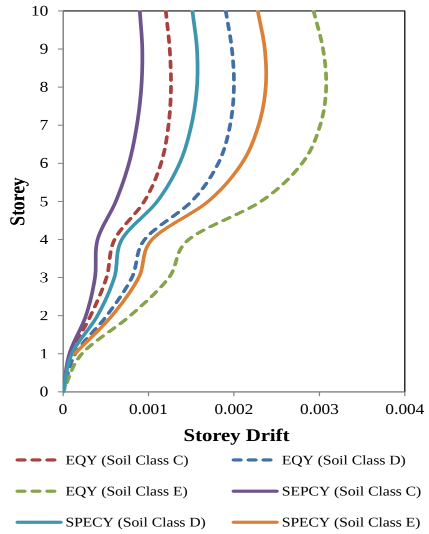




(a) Floor displacement profile for RSA analysis (1/1500 yrp) along the global Y-direction

(b) Storey eccentricity

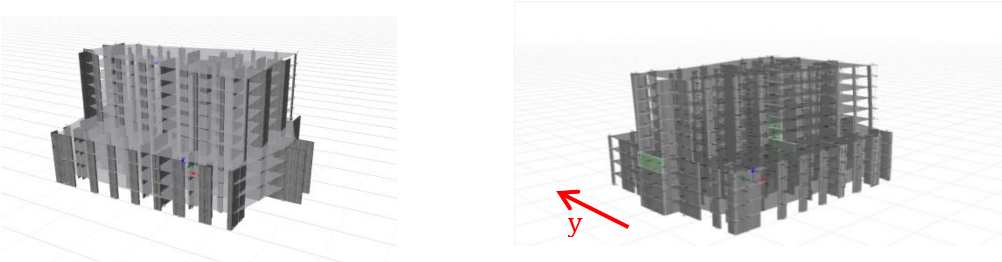
(c) Average storey stiffness (Y-direction) for RSA and EHL



(d) Storey Drift (1/1500 yrp) along global Y-direction
Figure 3: Linear analysis results

2.2 Nonlinear analysis (Pushover)

A full nonlinear pushover analysis was conducted to examine shear yielding and deformation patterns under quasi-static loading (pushover). Structural elements were discretised into inelastic beam-column displacement-based members modelled on SeismoStruct platform (SeismoSoft, 2007). Modal pushover analysis (Chopra and Goel, 2001) was conducted based on the fundamental mode shape obtained from eigenvalue analysis along two orthogonal directions (x and y).

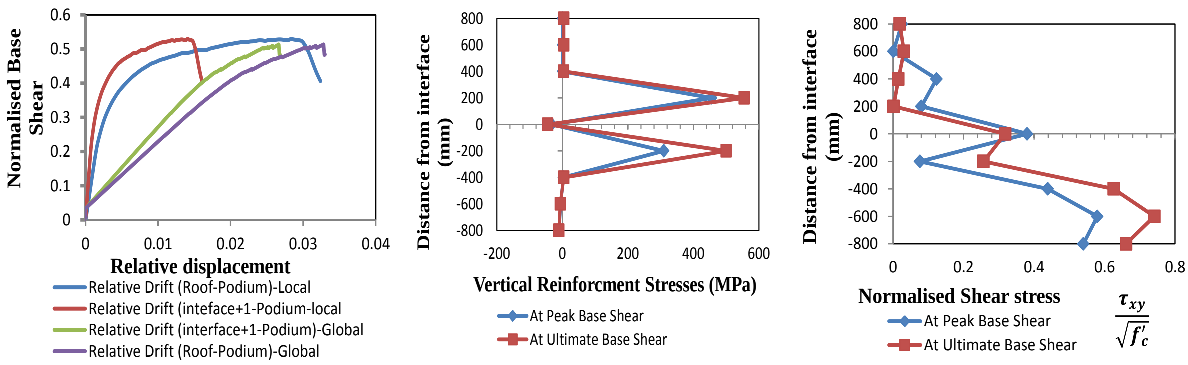


(a) 3D nonlinear model **(b) Shear-critical walls (pushover analysis-y)**

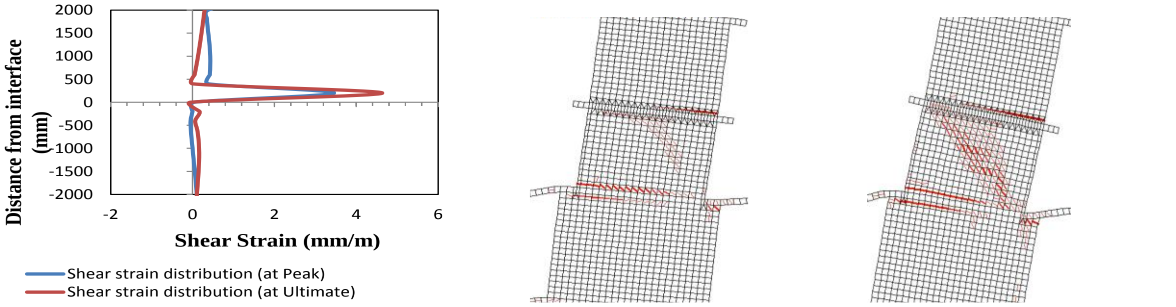
Figure 4: Nonlinear Model (SeismoStruct)

The following discussion pertains to pushover analysis along the global y direction. From the analysis, the elastic stiffness gradient between podium and tower levels was found to be 1.6. Highlighted walls in figure 4-b reached their respective shear capacity prematurely (prior to any significant flexural yielding). This can be explained with respect to earlier discussion on stiffness variation between the two portions of the structure, where the podium act as a rigid support that impose a stiff boundary condition on the tower, thereby increasing shear demands in the upper floors (directly above the interface). The latter is congruent with the “basement” effect phenomenon, where boundary conditions (above and below ground floor) impose higher shear forces in basement walls.

To investigate the implications of shear concentration on a local level, a reduced nonlinear FE model was assembled for the wall highlighted in figure 1-b on VecTor2; a two dimensional nonlinear finite element analysis program developed at the University of Toronto. Stiffness and strength distribution in the local (reduced) model was calibrated to match the global (full structure) model. Figure 5-a, plots the relative displacement profile (roof and level just above the interface with respect to podium level). The offset between the local and global simulations lies in the shear yielding mechanism, whereby in the former, sliding deformation across a base crack at the first level supressed the relative deformation between podium and upper floors up to a normalised shear force value of 0.45, the subsequent plateau marks the onset of successive horizontal cracking at the base of the tower walls, and consequently tipping off the relative displacement margin between the podium and tower levels. In the global analysis, the trend of behaviour was somehow different; this is primarily owed to the element formulation that builds up the structural system. With distributed plasticity models (used in SeismoStruct) the localised shear yielding is not explicitly enforced, such models often enforce strain compatibility and assume homogeneous yielding along the element, whereas in VecTor2, element formulation is based on the Modified Compression field theory (MCFT) (Vecchio and Collins, 1986) and Disturbed Stress field model (DSFM) (Vecchio, 2000), which are more suited for modelling shear-governed behaviour.



(a) Relative Displacement profile **(b) Stress in vertical reinforcement** **(c) Normalised shear stress distribution**



(d) Shear strain **(e) VecTor2 simulation of podium-interface crack pattern (at peak)** **(f) VecTor2 simulation of podium-interface crack pattern (at ultimate)**

Figure 5: Results from nonlinear analysis

Locally, at the podium-tower interface, inelastic excursions prompted localised horizontal cracks. This unfavourable mechanism is the direct result of abrupt vertical reinforcement curtailment (cut-off) above the podium level. Interestingly, this mode of failure was congruent to several wall failures reported in the post-earthquake reconnaissance following Christchurch 2011 earthquake (Kam et al., 2011). As shown in figure 5-b, vertical reinforcement stresses were concentrated within a distance of $\pm$ 200mm from the interface, further the difference between peak and ultimate load is only minor; highlighting the lack of internal redistribution of inelastic strains across the height of the wall. Figures 5-c and 5-d plot the normalised shear stress and shear strain along the vicinity of the interface respectively. Warping shear strains were found to be concentrated within a height of 500mm from the interface, implying profound sliding shear deformations were accumulated along the interface.

From the above study it can be shown that unfavourable shear failure mechanisms may be understated in conventional analysis/design procedures. In regions of low-to-moderate seismicity (such as Australia), explicit shear checks are often not required beyond strength bases(Wibowo et al., 2013, Wibowo et al., 2014). This was shown not to suffice in cases where global geometric irregularity imposes stringent demands, and consequently jeopardising structural seismic performance.

3 PROPOSED SHEAR CAPACITY ENVELOPE

In light of earlier case study example, shear-governed behaviour of walls is often understated in current practice. However localised, shear mechanism can substantially offset seismic performance of structures, and consequently result in non-ductile behaviour with minor seismic energy dissipation(Li et al., 2015). To better scrutinise seismic shear performance of walls, engineers and researchers are often assisted with conventional shear capacity models, these describe both strength and stiffness deterioration with increasing seismic demand. Most developed models (Kowalsky and Priestley, 2000, Krolicki et al., 2011, Salonikios, 2007), are based on rigorous experimental programs for ductile walls in different configurations and details. In this section, an alternative shear capacity envelope for limited ductile and lightly reinforced RC walls is proposed. The strain (damage) based model (though in the preliminary stages), is shown capable of predicting, with acceptable level of accuracy, the post-damage (post-peak) behaviour of walls failing in shear.

3.1 Rational model for shear strength degradation: Formulation

From first principles, a summary of the derivation of strain-based shear degradation model for low-ductility squat shear walls is described herein.

3.1.1 *Strain distribution in cantilever walls*

The longitudinal (flexural) and transverse (vertical) strains are denoted by ε'_c and $\varepsilon'_{c,ver}$ respectively (figure 6). From preliminary study on stress/strain distribution on walls under lateral loading, length of influence of $\varepsilon'_{c,ver}$ was shown to be proportional to the depth of neutral axis (c) and the longitudinal strain ε'_c:

$$\varepsilon'_{c,ver} = \alpha_\varepsilon \beta_\varepsilon \, \varepsilon'_c \qquad (1)$$

The coefficients α_ε, β_ε incorporate the effect of the magnitude of the applied load (vertical and lateral) and foundation/slab stiffness respectively. α_ε is expressed as:

$$\alpha_\varepsilon = \frac{F_{applied}}{\frac{M}{l_w/2} + P} \tag{2}$$

In essence, α_ε defines lateral load intensity with respect to vertical load (axial and resultant bending compression force). For lightly loaded cantilever walls, the expression can be simplified to:

$$\alpha_\varepsilon = \frac{F_{applied}}{\frac{M}{\frac{l_w}{2}}} = \frac{F_{applied}}{F_{applied} \times h_w \Big/ \frac{l_w}{2}} = \frac{1}{2a_s} \tag{3}$$

a_s is the aspect ratio of cantilever wall. The horizontal strain distribution is assumed to be localised along a specific height above the foundation (figure 6). The parameter β_ε encompasses the effect of relative stiffness of the foundation and wall on vertical strain distribution. It is speculated that additional confinement is provided to the lower portion of the wall through dilation stresses that arise from restrictive contribution of foundation/floor panels on the wall's toe. This was further examined by means of FE parametric study. Figure 9 shows the parametrisation of β_ε factor, it was noticed that with increase in the thickness of foundation/slab (t_f) with respect to wall thickness (t_w), vertical strains are alleviated relative to the longitudinal (flexural strains).

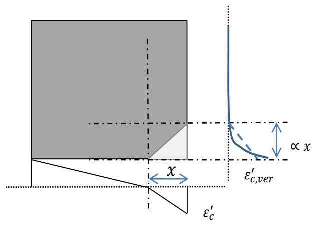


Figure 6: Vertical strain localisation

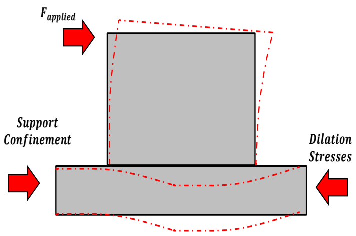


Figure 7: Schematic representation of dilation stresses under lateral loads

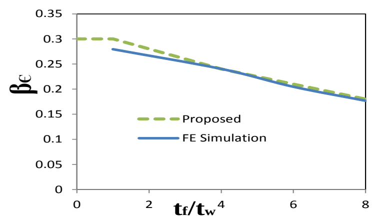


Figure 8: $\beta_\varepsilon$factor parametrisation

$$\beta_\varepsilon = \begin{cases} 0.3 & \frac{t_f}{t_w} \leq 1 \\ -0.015\frac{t_f}{t_w} + 0.3 \text{ for} & \frac{t_f}{t_w} > 1.0 \end{cases} \tag{4}$$

3.1.2 *Proposed shear degradation model for limited ductile RC walls*

Tip deflection due to vertical strain $\varepsilon'_{c,ver}$ could be found by integrating curvature distribution across the height of wall

$$\Delta_{Tip} = \int_0^{h_w} \varphi'_c z dz = \frac{\varepsilon'_{c,ver}}{\propto x}\left(\frac{h_w{}^2}{2}\right) \tag{5}$$

The depth of neutral axis can be expressed by rearranging the expression in (5)

$$x = \frac{\varepsilon'_{c,ver}}{\propto \Delta_{Tip}}\left(\frac{h_w{}^2}{2}\right) \tag{6}$$

The factor alpha describes the spread of strain localisation along the height of the wall, from the parametric study, $\propto$ and has been preliminary set to 1/3. A compression strut is assumed to transmit shear loads from point of application to the across compression node. Softened strut model (Hwang et al., 2001) was adopted to account for principle tensile strains normal to the compression field.

Compression softening parameter ς is given as:

$$\varsigma = \frac{5.8}{\sqrt{f_c'}} \frac{1}{\sqrt{1+400\varepsilon_r}} \leq \frac{0.9}{\sqrt{1+400\varepsilon_r}} \tag{7}$$

Where ε_r is the principle tensile strain along the principle direction. The strut geometry is deduced at the nodal zone by considering geometric compatibility at the node. Strut width is found to be:

$$\mathrm{w_{strut}} = \frac{\mathrm{x}}{\cos\theta_c} \tag{8}$$

At this stage, ς is predetermined as 0.4 as recommended by Vecchio and Collins (1986, 1993). From geometry and nodal equilibrium; the strut force can be formulated as:

$$\mathrm{R_d} = 0.4\mathrm{f}_c'\left(\frac{\mathrm{x}}{\cos\theta_c}\right)\mathrm{t_w} \tag{9}$$

Substituting kinematic relationship established earlier (equations 5 and 6) and defining ratio $\frac{\varepsilon_c'}{\varepsilon_{sy}} = \frac{0.003}{0.0025} \approx 1.2$; the softened concrete contribution to shear strength could be found in terms of displacement ductility demand:

$$\mathrm{V_c} = 0.18\frac{\mathrm{f}_c'}{\mathrm{a_s}^2}(\mathrm{t_w})\frac{\beta_e}{\propto \mu_\Delta}(\mathrm{h_w})\tan\theta_c \tag{10}$$

The above formulation provides a simplified analytical tool to estimate shear strength and stiffness degradation with increasing seismic demand (displacement ductility).

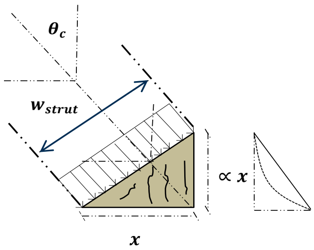


Figure 9: Schematic representation of compression node (STM)

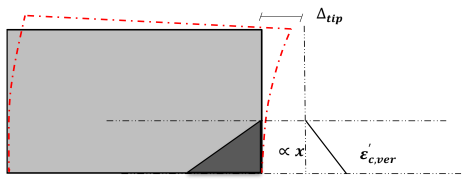
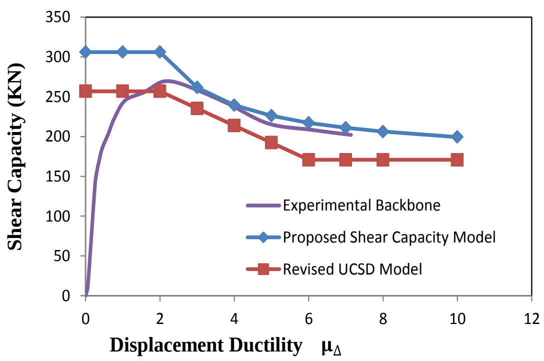


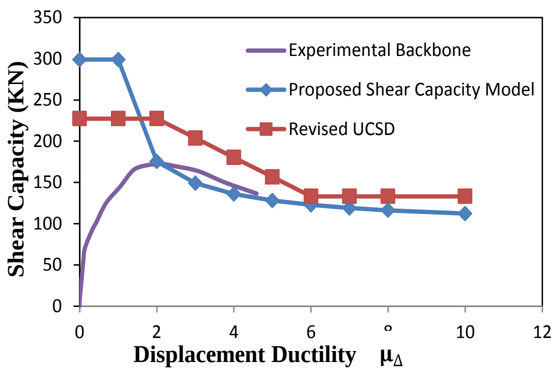
Figure 10: Vertical strain integration

3.1.3 *Model validation*

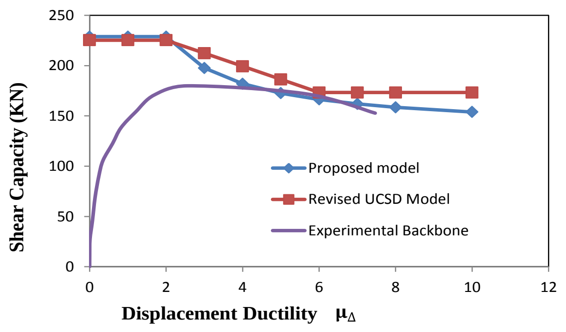
The above formulated shear strength degradation model has been validated by overlapping proposed capacity envelope on experimental force-displacement plots. Four representative walls were selected from the literature, namely: Salonikios LSW3 and MSW3 (1999) and Greinfenhagens'M3 and M4(2005). Revised UCSD (Kowalsky and Priestley, 2000) model was also superposed for the purpose of comparative analysis. Reinforcement detailing and material properties are reported elsewhere.

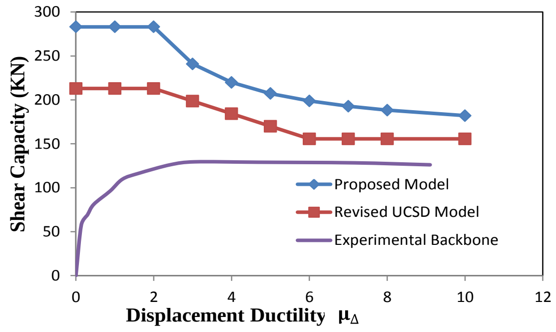


(a) LSW3



(b) MSW3





(c) M3 **(d) M4**

Figure 11: Proposed shear capacity envelope validation

Walls LSW3, MSW3 and M3 were all reported to have failed in flexural-shear, whereby flexural capacity is achieved prior to the onset of shear degradation. The model captured the post-damage behaviour (excursions following the onset of shear failure, denoted by the intersection of the capacity curve with the backbone) quite accurately, this is indicated by the consistent matching of descending backbone and proposed capacity curves. Wall M4 (figure 11-d) was reported to have failed in rocking/sliding across a prominent base crack; shear yielding was therefore not properly captured by the proposed model (nor by the UCSD model).

4 CONCLUDING REMARKS

This paper presented full linear and nonlinear analysis of buildings that adhere to the Australian norm for the purpose of illustrating shear deficiencies in some of the shear walls. Detailed nonlinear FE analysis of walls were modelled and assessed in terms of susceptibility to unfavourable and premature failure mechanisms. An alternative shear capacity envelope for walls designed and constructed in low-to-moderate seismicity regions is formulated and validated. The preliminary results have shown promising potential especially for walls failing in flexural-shear.

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