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SEISMIC ASSESSMENT OF NON-DUCTILE REINFORCED CONCRETE C-SHAPED WALLS

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Buildings that rely on reinforced concrete walls and cores as their primary lateral loading system are prevalent in much of Australia's building stock. Capacity design principles do not have to be adhered to in most low-to-moderate seismic regions, such as Australia. Consequentially, the level of detailing typically provided in accordance with the current and past concrete material standards, AS 3600 and AS 1480, is regarded as non-ductile from the seismic design point of view. These non-ductile reinforced concrete elements have been known to perform poorly when subjected to large lateral loads, as observed in the Christchurch earthquake in 2011. This paper presents an investigation into the seismic performance of C-shaped reinforced concrete walls acting as a core of a Mid-Rise building using current and past building codes in Australia. The displacement capacity of the building was calculated using a displacement-based assessment. A shear capacity model, which is a function of the curvature ductility of the walls, was also considered in the assessment. The results indicate that the older building is likely to fail in shear in the event of a 1000-year return period earthquake event. The building designed to current standards is vulnerable to a non-ductile failure from premature fracturing of the longitudinal reinforcing steel bars.

Keywords: U-shaped, Displacement-based, Shear, UCSD, AUS5, Low-to-moderate.

1 INTRODUCTION

Due to the low earthquake return period that is typically used in design, particularly for low-to-moderate seismic regions such as Australia, and the low standard of reinforcement detailing required by the current material provisions, it is likely that many reinforced-concrete (RC) walls and cores within these regions have very limited ductility (Hoult *et al.* 2014). It was the non-ductile RC structures that led to the majority of the loss of lives from the 2011 Christchurch Earthquake. Structural RC walls can be geometrically arranged in many ways. The channel-shaped, or C-shaped, is one of the simplest and is a popular arrangement used in practice (Beyer *et al.* 2008). Despite its popular use in industry, there have been relatively few studies on the inelastic behavior of RC-core structures (Beyer *et al.* 2008), warranting research on the seismic performance of C-shaped cores. This study considers two case studies in which RC walls and cores are the primary elements used in resisting lateral loads, including ground motions from a seismic event. They are both based on the same case study building, but one is designed in accordance with Australian Standards from the early 1980s, and the other in accordance with current Australian Standards.

2 CASE STUDY: MID-RISE BUILDING

A 5-story Mid-Rise RC office building, with a floor-to-floor height (h_s) of 3.5m, has a floor layout, as illustrated in Figure 1 (a). The columns have an 8.4-meter grid spacing, which is commonly used in office buildings in Australia. The dimensions of the 250mm thick (t_w) RC walls are given in Figure 1 (b).

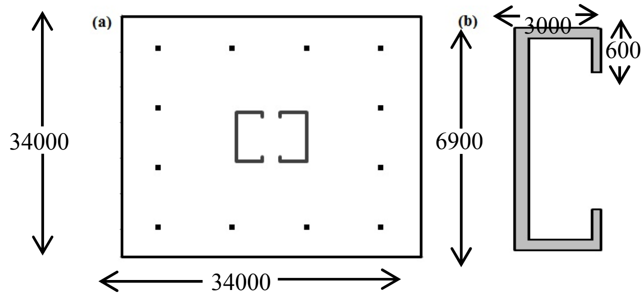


Figure 1. (a) Plan view of building with central core. (b) Dimensions of the C-shaped walls.

The dead load (G) and live load (Q) of the RC building are assumed to be 8 kPa and 4 kPa respectively. Two designs of the building have been undertaken; one building incorporates RC walls that have been designed with the current Australian building codes (Standards Australia 2009), the other with considerations of building codes that were required in the early 1980s (Standards Australia 1982). The building is assumed to be situated within the Melbourne CBD area and sited on soil class B_e (rock). The characteristic compressive strength of concrete (f'_c) is taken as 50 MPa, and the axial load ratio (ALR) on each wall was calculated to be 10%, based on resisting a floor area of 16.8m x 12.6m. The RC walls were initially designed for earthquake loading using AS 1170.4 (Standards Australia 2007) for the "2015 building", and the "1980s building" was designed for wind loading as per AS 1170.2 (Standards Australia 1983), as earthquake loading in design only became a requirement in the 1990s.

The base shear was found to be 203kN and 2695kN for the 1980s and 2015 buildings respectively, illustrating the significant difference in eras of design requirements. Torsional effects were also included in the design for the 2015 building by applying an eccentricity equal to 10% of the width of the building, as per Clause 6.6 of AS 1170.4 (Standards Australia 2007). This resulted in an increase of the base shear to 2971 kN and 4800 kN for directions that cause bending about the minor axis (Figure 2) and major axis respectively. The corresponding forces and moments were distributed to the C-shaped walls relative to their stiffness. This assumes that no other structural elements (e.g., RC frames) contribute to the seismic resistance of the case study buildings. Response-2000 (Bentz 2000) has been used to calculate the moment-curvature capacities of the walls. The detailing of the walls was chosen based on the requirements of the concrete structures code at the time of design – AS 1480 (Standards Australia 1982) and AS 3600 (Standards Australia 2009) for the 1980s and 2015 buildings respectively. Reinforcement was evenly distributed throughout the walls in two layers using Grade 230S and 500N deformed steel reinforcement bars for the 1980s and 2015 buildings respectively, as these are the grades of steel used at the time of design (CIA 2010). Figure 2 illustrates the governing strains for bending about the minor axis of C-shaped walls used in a core configuration. The failure of one wall (wall A) is governed by tensile strains, while the failure of the other (wall B) is governed by concrete strains, due to the geometrical configuration of the walls and direction of ground motion. The moment capacity about the minor axis, in which the tension strains are governing, tends to be the most critical for the moment capacity assessments.

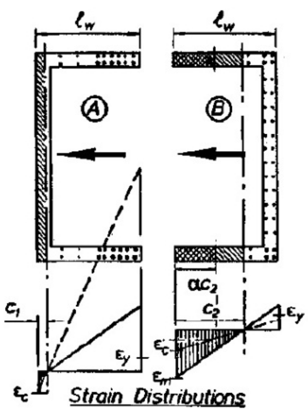


Figure 2. Strain distributions for the two walls subjected to bending in the same direction.

Using the relevant concrete structures code, the longitudinal (ρ_{wl}) and transverse steel reinforcement (ρ_{wt}) is uniformly distributed throughout the RC walls, with the corresponding ratios given in Table 1.

Building	ρ_{wl}	ρ_{wt}
1980s	0.21%	0.26%
2015	0.47%	0.34%

Table 1. Longitudinal and transverse reinforcement ratios for the walls.

The moments and curvatures from Response-2000 (Bentz 2000) were used in a displacement-based assessment (DBA) to calculate the displacement capacities of the C-shaped walls. The curvatures for bending about each of the two axes (about the major axis, and minor axis causing failures governed by tensile and concrete strains) are obtained for critical strain values that represent the onset of a particular performance objective. The performance objectives considered here are Serviceability, Damage Control and Collapse Prevention, which have the corresponding chosen conservative strain values of 1.5, 2, 3 for the concrete (ε_c) and 5, 10, 15 for the steel (ε_s) respectively in (mm/m). More information on the chosen strain values can be found in Hoult *et al.* (2014).

The displacements at the different performance objectives (or limit states) and for bending about the different axes were determined at the effective height (H_e) of an equivalent single-degree-of-freedom (SDOF) structure. The displacements at the onset of cracking and yield are also calculated and plotted. Initially the flexural behavior of the individual walls about the major axis and the minor axes in two directions have been assessed and compared with the shear capacities. When assessing the overall response of the building, the summation of force contributions from the two walls is carried out at the limiting displacements to calculate the total contribution to the resistance.

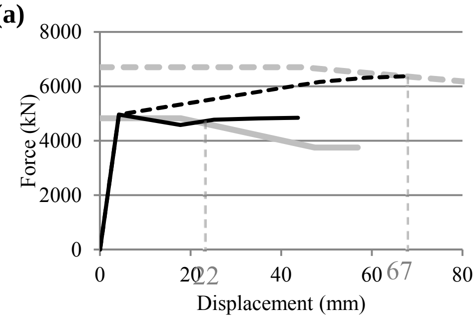
Recently, there have also been significant improvements in the understanding of the behavior of shear capacity in RC elements, including the loss of shear strength with repeated cyclic displacements. This is partly due to cracks forming in the plastic hinge zone at the base of the wall as the displacement of the wall increases, reducing the effectiveness of the interlocking effect of the aggregates along the cracked surface, and consequently reducing the shear resistance by the concrete. The modified UCSD shear model incorporated in Priestley *et al.* (2007) is used in the study to assess the shear capacity of the RC walls, which generally results in a higher shear capacity (for $\mu \leq 1$) compared to calculations from AS 3600 (Standards Australia 2009).

3 RESULTS AND DISCUSSION

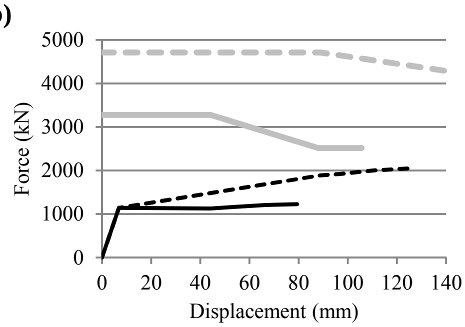
The results of the force-displacement relationship of the individual walls using DBA are illustrated in Figure 3(a-c) for bending about the major, minor (tension governing) and minor (compression governing) axes. Superimposed in these figures is the degrading shear strength results calculated using the proposed equations from Priestley *et al.* (2007). Figures 3(a) and 3(c) indicate that the walls can fail in shear when they bend about the major axis and minor (compression governing) axis. The new limiting displacements in Figures 3(a) and Figure 3(c) are 22mm and 18mm for the 1980s building and 67mm for the 2015 building, as illustrated in the figures. The large cracking force (F_{cr}) in Figure 3(c) is due to a higher cracking moment (M_{cr}) in comparison to the ultimate moment capacity (M_u) of the wall.

The corresponding displacement capacities, in the form of displacement response spectrum, are shown in Figure 4 for ground motions that cause bending about the cores (a) major axis and (b) minor axis. Note that the markers (circles) indicate the limiting displacements due to shear failure.

(a)



(b)



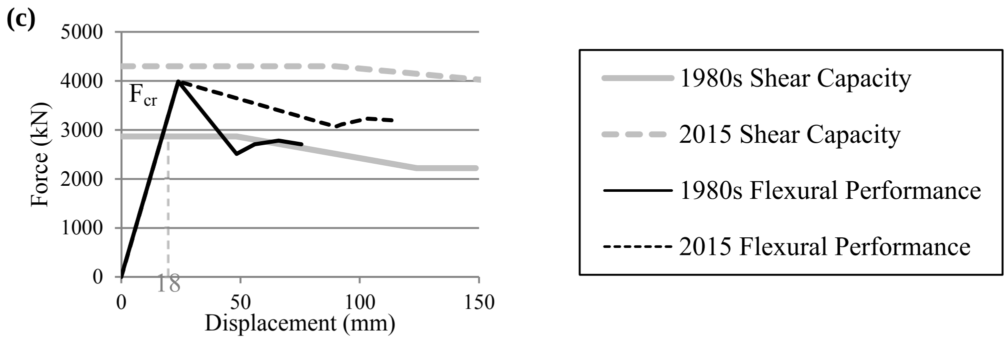


Figure 3. Displacement capacity of the individual C-shape wall for bending about the (a) major (b) minor (tension governing) and (c) minor (compression governing).

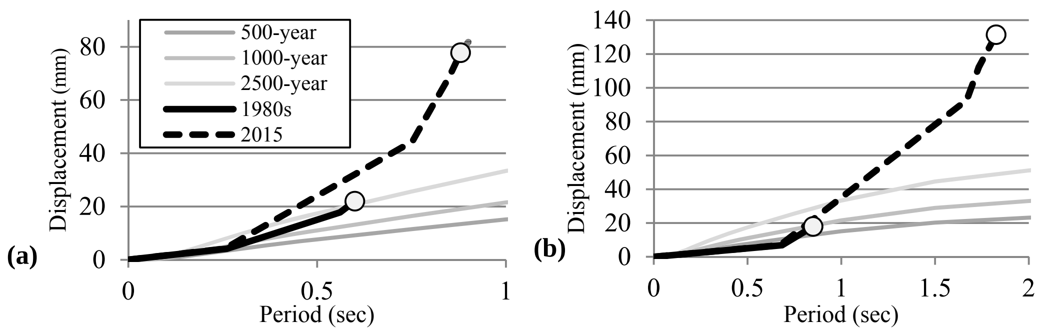


Figure 4. Displacement capacity of the buildings for bending about the (a) major axis and (b) minor axis and demand for different earthquake return periods.

Superimposed on Figure 4(a) and Figure 4(b) are the displacement demand for 500, 1000 and 2500 year return period based on a Probabilistic Seismic Hazard Analysis (PSHA) using the AUS5 earthquake recurrence model (Brown and Gibson 2004). Figure 4(b) indicates that the RC walls incorporated in the 1980s building could fail under 1000 year return period ground motions. However, the UCSD shear degradation model used from Priestley *et al.* (2007) assumes that there will be a distribution of cracks in the "plastic hinge region". Observations from the Christchurch Earthquake event have suggested that lightly reinforced walls, such as the walls investigated here, will generally form a single crack at the base of the wall and all of the plasticity to occur over this very short length of the wall height (CERC 2012). This is further suggested to occur when M_{cr} is larger than M_u, as was the case for the walls in both building cases. The suggestions from Henry (2013) indicate that the even the RC walls designed to current standards will fail due to premature fracturing of the longitudinal reinforcing bars. This will be the focus of a future study.

4 CONCLUSIONS

This investigation of an old mid-rise building, incorporating C-shaped walls with typical values used in low-to-moderate seismic regions such as Australia, reveal the potential seismic vulnerability of the structural elements with pre-emptive shear failure. Further, the RC walls may have insufficient longitudinal reinforcement, which restrict the wall from forming secondary cracking, as indicated by a higher M_{cr} than M_u.

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