Seismic Performance of Typical C-Shaped Reinforced Concrete Shear Cores in Australia

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ABSTRACT:

Limited-ductile reinforced concrete structures have been known to perform poorly when subjected to large seismic ground motions. Many buildings in Australia rely on reinforced concrete shear-cores as their primary lateral load resiting system, but these are only required to have a low standard of detailing as per the current concrete material standards AS 3600. While there is some literature available on the numerical and analytical modelling of rectangular shaped shear walls, non-rectangular shear walls have not been extensively analysed. "C-shaped" shear walls are commonly found enclosing a service core, lifts, stairs and toilets. This paper presents a study which looks at the seismic performance of C-shaped shear walls with different steel reinforcement ratios for low, mid and high-rise buildings. The current earthquake actions code AS 1170.4 has been used for a preliminary design of the walls using a force-based design approach within which the intention is to satisfy the performance objective of life safety in a 500 year return period earthquake design level event in Melbourne. The displacement capacity of these different core walls has been calculated using a Displacement-Based Assessment procedure and complemented with finite element modelling program SeismoStruct. The results of a probabilistic seismic hazard analysis (PSHA) using the AUS5 recurrence model that has been conducted for the city of Melbourne has been used to calculate more accurate predictions of the displacement response spectra for 500 and 2500-year return periods. The C-shaped core walls have then been assessed by comparing the displacement capacity at different structural performance limit states to the displacement spectra derived from the current earthquake loading code AS 1170.4 and the spectra results from the PSHA for both the 500 and 2500-year return periods and for soil classes B_e and D_e.

Keywords: Walls, Core, displacement-based, low-to-moderate, seismicity, capacity, Australia, earthquake, spectra, AUS5, PSHA, concrete, limited-ductile, unconfined

1. Introduction

Reinforced concrete (RC) cores are used throughout Australia's building stock, with many of the Low-Rise, Mid-Rise and High-Rise structures relying on these elements to resist the primary lateral loads the building is subject to, including earthquake ground motions. Not only do these cores provide a significant percentage of the lateral load resistance but the core is often used for accommodating the lift shafts or stair cases (Beyer *et al.*, 2008). Due to the low earthquake return period used in design and the low standard of detailing required in the current material standards in Australia, it is anticipated that most of these RC cores embedded within structures around Australia are "limited-ductile". 'The underlying philosophy of the earthquake loading standard [AS 1170.4] is to protect life by preventing building collapse whilst accepting that significant damage could occur' (Wilson *et al.*, 2008). However, as observed in Christchurch, it is the very rare earthquake event, with ground shaking reaching or exceeding the 2500-year return period event (Goldsworthy, 2012), that has the potential to cause major destruction in the regions of low-to-moderate seismicity, such as Australia, where injuries and possibly deaths are expected given the vulnerability of some of the structures.

Of the many possible shapes of structural cores, the channel-shaped (C-shaped), sometimes referred to as U-shaped, is one of the simplest and most used (Beyer *et al.*, 2008). Despite its popular use in practice there have been relatively few studies on the inelastic behaviour of RC core structures (Beyer *et al.*, 2008). It is therefore desirable to investigate the performance of limited-ductile C-shaped RC walls in the event of a very rare earthquake.

2. Building and C-shaped Core layouts

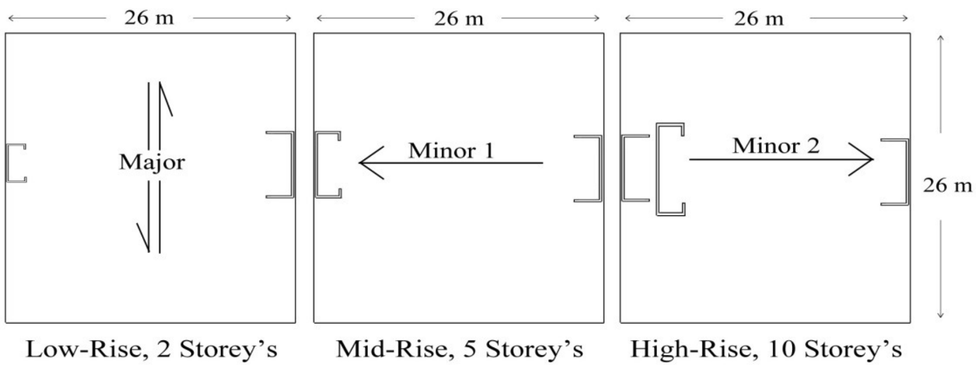

Figure 1 Floor layouts for the three building types with open and partially closed cores

The building footprint considered for this study is shown in Figure 1, which has been kept at a constant area of 676 m^2. The directions of motion that are to be considered are shown in Figure 1 for bending about the minor axes of the various walls. Bending about the major axes will also be considered. The open and partially closed core shapes shown in Figure 1 will be used in representing the stair and lift C-shaped walls respectively. The Low (2 storey) and Mid-Rise (5 storey) buildings consist of one elevator and stair core, while the High-Rise (10 storey) building will be complimented with another stair core, as shown in Figure 1. The dimensions of the stair core, used for all buildings, and the lift cores for the different rises are given in Table 1, where the symbols are given in the Appendix. These floor layouts and wall dimensions have been based on BCA guidelines (BCA, 2008), typical lift sizes and number of cars required as indicated in RLB (2014), as well as consultations with people in industry. The low-rise, mid-rise and high-rise buildings will have a total of 2, 5 and 10 stories respectively, corresponding to the definitions of the different rises given in HAZUS (FEMA, 2003) for concrete shear wall buildings. The seismic mass ($G + \psi Q$) is assumed to be 8 kPa, characteristic compressive strength of concrete (f'_c) of 40 MPa, the floor-to-floor height (h_s) is 3.5 m, and the axial load acting on each of the core walls is assumed to be 25% of the floor weight.

Table 1 Dimension of gross cross-sectional area of C-shaped walls

Rise	Wall Type	l_w (mm)	b (mm)	t (mm)	w (mm)
Low	Lift	3300	1850	150	300
Mid	Lift	6300	2650	200	400
High	Lift	8600	2650	250	1200
Low, Mid, High	Stair	6300	2650	200	-

The seismic shear force and moment demands are calculated for each of the buildings using the horizontal equivalent static procedure as per clause 6.2.1 in AS 1170.4 (Standards Australia, 2007) for soil classes B_e and D_e. Wind actions have also been considered, as per AS 1170.2 (Standards Australia, 2011), however the base shear calculated from earthquake actions is found to dominate for all of the buildings considered. These forces and moments are then distributed to the lift core and staircase core relative to their stiffness in each of the three directions being considered. The results of the shear and moment demand for the two different soil conditions and different configurations are given in Table 2. The fundamental natural period (T_1) of the buildings are calculated using clause 6.2.3 of AS 1170.4 (Standards Australia, 2007). The minimum reinforcement detailing for each of the different wall configurations is then calculated to ensure that the capacity is higher than the initial estimates of demand. For ease of construction it is preferable that the steel reinforcement is distributed throughout the concrete sections evenly spaced, however this was not possible with some of the higher moment demands found for taller buildings on softer soils, and particularly for bending about the minor axes. The moment-curvature analyses for the walls were calculated with RESPONSE-2000 (Bentz, 2000). The results and details of the minimum steel reinforcement ratio required for each of the walls in different rises and for the two soil classes are given in Table 3. The vertical reinforcement ratio (ρ_{wv}) is kept constant up the height of the walls, and so it is assumed that the plastic hinge will form at the base of the walls. It is found that, even with the minimum required amount of transverse (horizontal) reinforcement (ρ_{wh}), all of the walls have sufficient capacity in resisting the initial shear demand, as shown in Table 3 and calculated based on clause 11.6 of AS3600 (Standards Australia, 2009). It should also be noted that M^* and ΦM_u in Table 3 have only been given for bending about one of the directions of the minor axis for each wall, which corresponds to the least moment capacity necessary for bending about that axis (weakest direction of motion).

Table 2 Shear and moment demand

Rise	T_1 (sec)	Soil Classification	V_b (kN)	M (kNm)
Low	0.27	B_e	979.4	5141.9
Low	0.27	D_e	1225.9	6436.1
Mid	0.53	B_e	1382.8	18149.4

Mid	0.53	D_e	3064.8	40225.8
High	0.9	B_e	1628.6	42752
High	0.9	D_e	3664.5	96192.1

Table 3 Reinforcement ratio required in the C-shaped walls

Rise	Wall Type	Soil Class	V* (kN)	ΦV_u (kN)	M* (kNm)	ΦM_u (kNm)	ρ_{wv}	ρ_{wh}
Low	Elevator	B_e	212.0	1110.3	1113.1	2421.9	0.29%	0.25%
	Stair	B_e	767.4	2313.6	4028.8	5771.5	0.72%	0.25%
Low	Elevator	D_e	265.4	1110.3	1393.3	2421.9	0.29%	0.25%
	Stair	D_e	960.5	2313.6	5042.8	5771.5	0.72%	0.25%
Mid	Elevator	B_e	749.9	2313.6	9842.9	14643.6	0.73%	0.25%
	Stair	B_e	632.9	2313.6	8306.5	8672.0	0.99%	0.25%
Mid	Elevator	D_e	749.9	2313.6	21815.6	22596.0	1.04%	0.25%
	Stair	D_e	1402.7	2313.6	18410.2	18986.4	1.63%	0.25%
High	Elevator	B_e	885.4	2892.0	23242.5	26062.0	0.66%	0.25%
	Stair	B_e	371.6	2313.6	9754.8	15624.8	0.98%	0.25%
High	Elevator	D_e	1992.2	2892.0	52295.7	53820.1	1.35%	0.25%
	Stair	D_e	836.1	2313.6	21948.2	22056.8	1.63%	0.25%

3. Performance Objectives

The critical strain values for different performance objectives given in Priestley *et al.* (2007) for well confined concrete have been modified for use in assessing the performance of walls with non-ductile detailing. For well-confined concrete the transverse ties (horizontally) and longitudinal reinforcement (vertically) are closely spaced; not only is a higher concrete axial strain allowable, but also the longitudinal bars are well restrained and less likely to buckle when the outer concrete spalls off. For the purpose of this study strain limits have been chosen for the concrete and steel at each of three structural performance limit states; these are given below. Given the uncertainty in the predication of displacement demand (Leonard *et al.*, 2014) and the lack of experimental data on the cyclic behaviour of non-ductile walls, conservative values have been chosen here. Consideration has been given to the brittle nature of possible failure modes such as bar buckling, out-of-plane buckling of walls, concrete crushing and low-cycle fatigue of bars, which can occur 'at levels of tensile strain significantly below ε_{su}' (Priestley *et al.*, 2007).

Serviceability (ε_c=0.001 and ε_s=0.005): The concrete stress-strain curve is close to linear and steel strains limited to twice the nominal yield value so that residual crack widths are small.

Damage Control (ε_c=0.0015 and ε_s=0.01): Concrete is now in non-linear range but there is a low expectation of spalling. Steel strains are sufficiently low so that repair is inexpensive; Also, there is low likelihood of low cycle fatigue or out-of-plane buckling on load reversal.

Collapse Prevention (ε_c=0.002 and ε_s=0.015): Margin is given against spalling (typically expected at a concrete strain of 0.003 or 0.004) due to the very brittle nature of the potential failure (crushing and longitudinal bar buckling). Steel strains are limited to prevent collapse due to low cycle fatigue (due to inelastic cycles in main event plus aftershocks) and out-of-plane buckling on reversal of load.

The results for the strain values for each of the walls obtained from the section analysis program RESPONSE-2000 (Bentz, 2000) show that generally the steel strains dominate (or occur before the concrete limit strains) for the C-shaped sections about the major and one of the minor axes, depending on the direction of motion. This is shown in Figure 2, where the minor 1 direction for the High-Rise building has two walls dominating in tension, whereas the other wall is dominating in compression in this direction due to its orientation. Therefore, the concrete strains dominated the performance for the C-shaped walls about the other minor axis for all of walls reflected in Figure 2. This was particularly true for some of the more heavily reinforced walls, which were yet to reach yield in the longitudinal steel (ε_s=0.0025) when the concrete "Collapse Prevention" strain is reached.

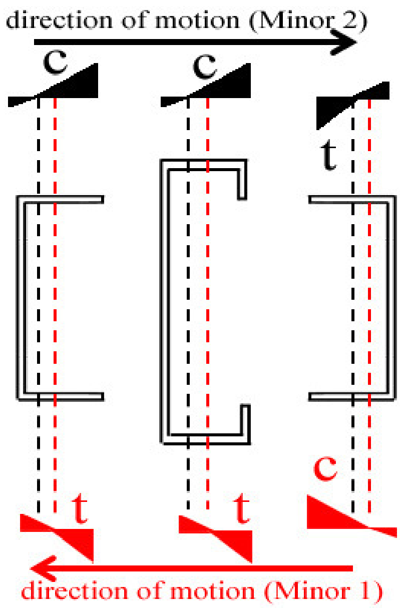

Figure 2 Plan view, showing directions of motion (Minor 1 and 2) for the High-Rise building wall orientations with bending about the neutral axis (dotted line) causing the walls to be dominated by tension (t) or compression (c)

4. Displacement-Based Assessment

A displacement-based assessment (DBA) of the walls was carried out to find the buildings' displacement capacities. The limiting curvature (and corresponding moment) at the base of each wall corresponding to the three different performance limit states has been calculated about the major axis and each of the minor axes using RESPONSE-2000 (Bentz, 2000). The limiting displacements at the effective height (H_e) of an equivalent single-degree-of-freedom (SDOF) structure were then determined for each wall about each axis and for each limit state using Equations 1 or 2 from Priestley *et al.* (2007). It should be noted that Equations 1 and 2 are dependent on the strain penetration length (L_{sp}) and the plastic hinge length (L_p), which were calculated using the empirically derived equations given in Priestley *et al.* (2007) with

Equations 3 and 4. These equations may not be valid for unconfined concrete walls , and further research is necessary to determine the plastic hinge length of lightly reinforced walls.

$$\Delta_l = \emptyset\left(H_e + L_{sp}\right)^2/3 \qquad \text{if } \varepsilon_s \leq \varepsilon_y \tag{1}$$

$$\Delta_l = \Delta_y\left(\frac{M}{M_y}\right) + \left(\emptyset - \emptyset_y\left(\frac{M}{M_y}\right)\right)L_p H_e \qquad \text{if } \varepsilon_s > \varepsilon_y \tag{2}$$

$$L_{sp} = 0.022 f_{ye} d_{bl} \tag{3}$$

$$L_p = k.H_e + 0.1 l_w + L_{sp} \tag{4}$$

where the symbols representing the different parameters are found at the back in the Appendix. For a given direction of earthquake loading, the minimum displacement of each RC wall in the building and for each performance state was found, which corresponded to the limiting displacement (Δ_l) for that performance state. The lateral force corresponding to this limiting displacement was calculated for each wall using Equation 5. In this equation, M_{limit} is the moment corresponding to the limiting displacement (Δ_l). The summation of the forces from all of the walls, due to the contribution of resistance, at this displacement is equal to the base shear at that limit, $V_{b.limit}$.

$$F_{limit} = M_{limit}/H_e \tag{5}$$

In order to compare the displacement capacity at the three critical performance levels to the displacement demand, the stiffness and period of the structure need to be calculated for each limit state and direction of motion using Equations 6 and 7:

$$K_{limit} = V_{b.limit}/\Delta_l \tag{6}$$

$$T_{limit} = \sqrt{\frac{4\pi^2 m_e}{K_{limit}}} \tag{7}$$

The force versus displacement results from the displacement based analysis have then been verified with the finite element modelling program SeismoStruct (SeismoSoft, 2013) using a pushover analysis. An example of the resulting force-displacement plot comparing the DBA results for the High-Rise building sited on soil class B_e to the results from SeismoStruct is shown in Figure 3. These results are for the direction of motion to cause bending about the Minor 1 axis, as shown in Figure 1.

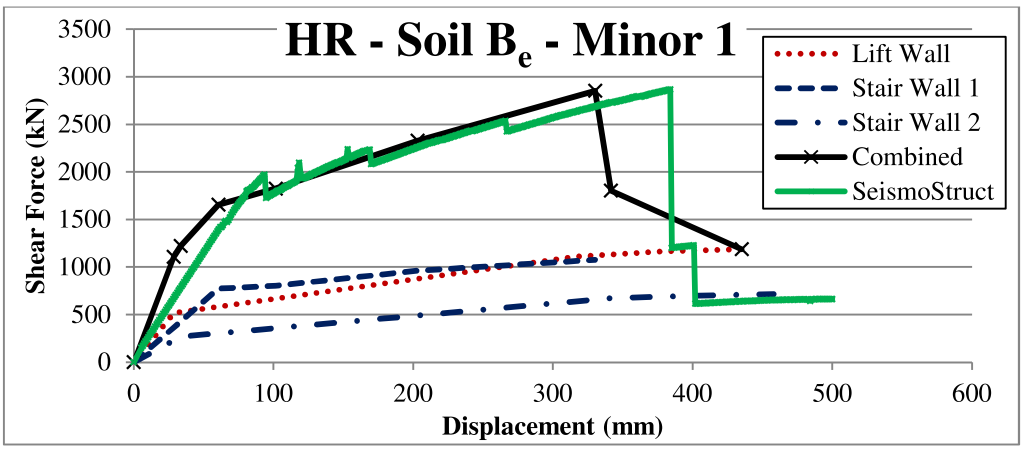

Figure 3 Displacement capacity of the Mid-Rise (MR) building on soil class D_e

5. Assessment against a 2500-year return period earthquake event

Using the DBA results, it is possible to compare the critical displacement capacities of the walls with the demands determined using appropriate displacement spectra. The response spectrum for a 2500-year return period in Melbourne has been derived from a probabilistic seismic hazard analysis (PSHA) using the AUS5 seismotectonic recurrence model from Brown and Gibson (2004). Further details on the seismic hazard derivation can be found in Hoult (2014). The PSHA displacement spectra results for a 2500-year return period earthquake event on rock B_e (V_{S30} = 760 m/s) and on site class D_e in Melbourne for 5% damping are shown in Figure 4; the spectra derived from AS 1170.4 are also superimposed. Figure 4 illustrates that the AUS5 results give increasing displacement response in contrast to the AS 1170.4 spectra, which has a "cut off" displacement at the corner period of 1.5 seconds. The increasing response from AUS5 is a result of the ground motion prediction equations used in the PSHA to derive the response spectra, which account for a wide range of deep soils and soft rock and correspondingly with resonance occurring at a range of periods (Hoult *et al.*, 2013). The AUS5 results are also more representative of the increasing displacement response observable from recordings, further shown in Amirsardari *et al.* (2014). The enforced "cut-off" second corner period value of 1.5 seconds is also the subject of some scrutiny (Hoult, 2014). The displacement spectra was used to find the displacement demand values at T_{limit}. It should be noted that if the building has been loaded past the yield limit then the corresponding damping value should be higher than 5%, which has been used for the displacement spectra and hence the displacement demand predicted may be conservative.

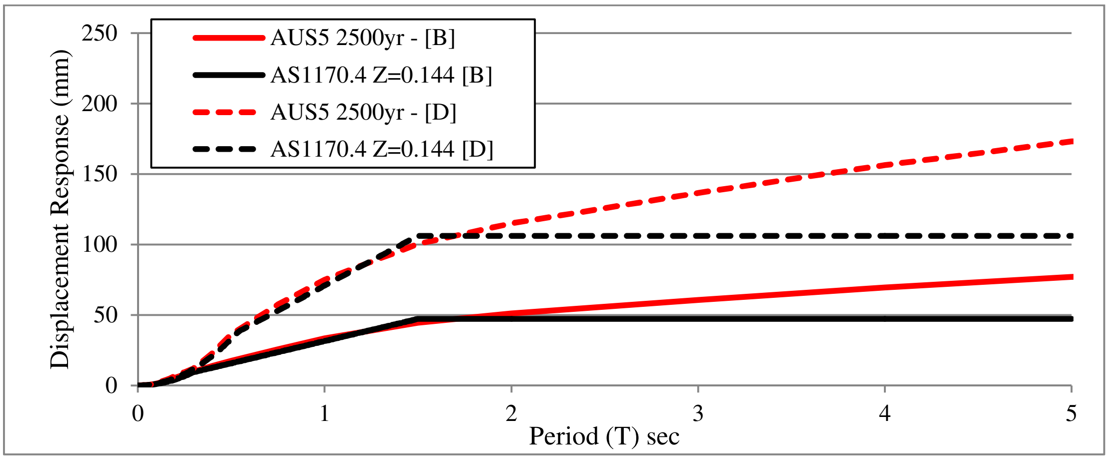

Figure 4 Displacement Response (5% damping) for 2500-year return periods and soil class B_e and D_e

Tables 4 to 6 give the results for the different buildings investigated on the two different soil types and indicate whether they reach or exceed a particular performance limit state for a 500 and 2500-year return period earthquake in Melbourne. Both the AUS5 and AS 1170.4 displacement response spectra are used, where "N" indicated that the limit state is not reach and "Y" indicates that it is. For a 500-year return period earthquake event in Melbourne the "Damage Control" performance state was only reached or exceeded by the Low-Rise buildings on soil class D_e and not by the AUS5 spectrum. The results from the 2500-year return period spectra indicate poor performance in several situations. Both of the displacement spectra indicate that the Low-Rise building on soil class D_e will reach or exceed the "Collapse Prevention" limit state. The results are similar for the Mid-Rise building on soil D_e; both spectra indicate that the "Damage Control" limit state is expected to be reached or exceeded in this event. The 2500-year return period event is only expected to cause minor damage to the High-Rise buildings, with indication of the "Serviceability" performance limit state being reached or exceeded for these buildings on soil class D_e.

Table 4 The performance limit states reached or exceeded for the Low-Rise buildings

	500-year Return Period (AUS5)			2500-year Return Period (AUS5)		
Soil Class	**Serviceability**	**Damage Control**	**Collapse Prevention**	**Serviceability**	**Damage Control**	**Collapse Prevention**
B	N	N	N	Y	N	N
D	Y	N	N	Y	Y	Y
	500-year Return Period (AS 1170.4)			**2500-year Return Period (AS 1170.4)**		
Soil Class	**Serviceability**	**Damage Control**	**Collapse Prevention**	**Serviceability**	**Damage Control**	**Collapse Prevention**
B	N	N	N	Y	N	N
D	Y	Y	N	Y	Y	Y

Table 5 The performance limit states reached or exceeded for the Mid-Rise buildings

	500-year Return Period (AUS5)			2500-year Return Period (AUS5)		
Soil Class	**Serviceability**	**Damage Control**	**Collapse Prevention**	**Serviceability**	**Damage Control**	**Collapse Prevention**
B	N	N	N	N	N	N
D	N	N	N	Y	Y	N
	500-year Return Period (AS 1170.4)			**2500-year Return Period (AS 1170.4)**		
Soil Class	**Serviceability**	**Damage Control**	**Collapse Prevention**	**Serviceability**	**Damage Control**	**Collapse Prevention**
B	N	N	N	N	N	N
D	Y	N	N	Y	Y	N

Table 6 The performance limit states reached or exceeded for the High-Rise buildings

	500-year Return Period (AUS5)			2500-year Return Period (AUS5)		
Soil Class	**Serviceability**	**Damage Control**	**Collapse Prevention**	**Serviceability**	**Damage Control**	**Collapse Prevention**
B	N	N	N	N	N	N
D	N	N	N	Y	N	N
	500-year Return Period (AS 1170.4)			**2500-year Return Period (AS 1170.4)**		
Soil Class	**Serviceability**	**Damage Control**	**Collapse Prevention**	**Serviceability**	**Damage Control**	**Collapse Prevention**
B	N	N	N	N	N	N
D	N	N	N	Y	N	N

6. Conclusion

Case study buildings of 2 (Low-Rise), 5 (Mid-Rise) and 10 (High-Rise) storeys with realistic lateral force-resisting systems consisting of C-shaped walls and the same plan dimensions have been assessed here by examining their flexural displacement capacity and comparing it with the displacement demand found from spectra obtained from AS 1170.4 and from the AUS5 model (both 500 and 2500 year return periods). The results are preliminary ones only since certain assumptions made in the calculations still need to be verified, which include the plastic hinge length, the values chosen for the strain limits, and the soil amplification factors from AS 1170.4 used to find the displacement spectra for the class D_e site (see Amirsardari *et al.*, 2014). The strain limits were chosen conservatively due to the uncertainty in the prediction of the displacement demand and the lack of experimental data on the cyclic behaviour of non-ductile walls. Other failure modes have been neglected for this research, such as shear, torsion and excessive drifts, and the focus has been on the failure of the walls through flexurally dominated behaviour. It should be noted, however, that due to flexural overstrength, especially for bending about the major axis, the shear forces reached are likely to be higher than the design shears, i.e. the values given in Table 3. Furthermore, AS 1170.4 requires earthquake design actions in orthogonal directions; that is, taking 100% of the horizontal earthquake forces for one direction and 30% in the orthogonal direction. The author concedes that this may influence some biaxial bending of the cores, ultimately impacting on the final conclusions drawn here, and will be investigated with additional studies. Further research is being conducted at the University of Melbourne on assessing the performance of structural walls against a "very rare" earthquake, which include the use of a range of parameters and the effects of other actions, such as shear, torsion and excessive drifts

7. Appendix

Symbols List

ALR	axial Load Ratio
b	breadth of wall (flange length)
B_e	soil class (Rock)
$C_h(T)_b$	Spectral Shape Factor on rock at a particular period
$C_h(T)_d$	Spectral Shape Factor on deep or soft soil at a particular period
d_{bl}	diameter of the longitudinal reinforcement
d	depth of wall (web length)
D_e	soil class (deep or soft soil)
F	force in kilonewton
f_u	steel ultimate stress
f_{ye}	yield strength of the longitudinal reinforcement
f'c	characteristic compressive (cylinder) strength of concrete at 28 days
G	dead load
H_e	effective height
h_s	interstorey height
k	$0.2(f_u/f_y -1) \leq 0.08$
K	stiffness
L_p	plastic hinge length
L_{sp}	strain penetration length
l_w	length of wall
M	moment in the wall at a particular limit state
M*	moment demand

m_e	effective mass
M_u	ultimate moment capacity
M_y	moment at yield
Q	live load
S_T	soil response amplification factor
T	period (s)
t	thickness of wall (flange and web)
T_{limit}	Period (s) corresponding one of the performance limits
V_b	base shear force
V_{S30}	shear-wave velocity in the top 30 meters
w	length of the "boundary" element of the partially closed core
Δ	displacement
Δ_y	displacement at yield
ε_c	strain of concrete
ε_s	strain of steel
ε_{su}	ultimate strain of steel
ε_y	yield strain in steel
ρ_{wv}	longitudinal reinforcement ratio
φ	curvature
Φ	reduction factor, taken as 0.8
φ_y	curvature at yield
ψ	earthquake imposed action combination factor

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