Changes in Seismic Design Philosophy for RC Structures in Areas of Low to Moderate Seismicity Following the Christchurch Earthquake

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SUMMARY:
The vulnerability of low to medium rise reinforced concrete buildings in Australia that use non-ductile reinforced concrete structural walls as the primary lateral force-resisting system has been assessed for a 475 year and 2475 year return period earthquake hazard for buildings sited in the central business district (CBD) of the capital city of South Australia, Adelaide, on deep and/or soft soil. This study illustrates a key lesson from the Christchurch earthquake sequence in which two non-ductile reinforced concrete buildings (the Pyne Gould and the CTV buildings) that functioned well in the "Darfield" earthquake in September 2010 (an approximately 475 year return period event) then failed catastrophically in the approximately 2475 year return period (February 22nd 2011) event. This study considers a realistic range of wall parameters and investigates which types of walls are the most vulnerable in a very rare event.

Keywords: *Seismic Design Philosophy, Non-ductile reinforced concrete walls, displacement-based assessment*

1. INTRODUCTION

There has been considerable debate in Australia about the adequacy of the return period used for the design level earthquake. In the commentary to the latest version of the earthquake loading standard (Standards Australia, 2007) the comment is made in section C3.1 that the U.S. and Canada have moved from a 500 year return period to a 2500 year return period and that "the difference between these return periods is much more significant for low to moderate seismic regions typically associated with intraplate earthquake activity such as Australia". (In fact the U.S. at the time had a design level earthquake (a rare event) at two-thirds of that corresponding to the 2500 year return period event and the 2500 year return period event (a very rare event) was nominated to be the maximum considered event (MCE). Nevertheless, the basic point remains that the use of a 500 year return period to determine the design level earthquake has been questioned.)

In Australia low to medium rise reinforced concrete buildings typically rely on a core that consists of two walls in each direction to withstand the loads due to wind and earthquake forces. The core is usually placed centrally in the building. Reinforced concrete frames in buildings with substantial core walls, although moment resisting, are typically considered more flexible than the core walls, and are not usually designed for lateral forces (and are not required to be detailed to accommodate the interstorey drifts due to earthquake response). The type of detailing used in these reinforced concrete buildings has limited ductility and AS3600, the Australian Standard for concrete structures (Standards Australia, 2009), does not require designers to use capacity design principles; i.e. there is no requirement to consider strength hierarchies and the provision of ductile energy dissipating mechanisms for seismic design. Designers usually use a force-based approach to design, with the design level earthquake forces determined by carrying out an equivalent static analysis in accordance with Chapter 6 from the earthquake loading standard, AS1170.4 (Standards Australia, 2007). The return period for the design level earthquake is based on the "Importance Level" of the building and this is specified in the Building Code of Australia (ABCB, 2011). Most ordinary buildings are of Type 2 and the BCA specified return period for the Australian standards-based ultimate design is 500 years.

In the Christchurch (February 22nd, 2011) earthquake some non-ductile reinforced concrete buildings performed poorly, with two of them, the CTV and the Pyne Gould buildings, failing catastrophically (Goldsworthy, 2012). The ground motions experienced in that earthquake generated displacement responses in the Christchurch CBD in excess of the codified 2475 year return period event for Christchurch over a wide range of periods relevant to building structures. In this paper, displacement-based assessments are used to determine the vulnerability of low to medium rise reinforced concrete buildings that use non-ductile reinforced concrete walls as the lateral force-resisting system. The vulnerability will be assessed for the 475 year and 2475 year return period level of hazard in the Adelaide CBD. Adelaide, the capital city of South Australia, has been chosen for this case study since it has one of the highest degrees of seismic hazard of the capital cities in Australia and also has unfavourable deep and/or soft soil at many sites in the CBD.

Displacement spectra have been developed for this study using the latest techniques available. As in most countries, early developments in earthquake risk mitigation in Australia used an earthquake hazard criterion to act as a proxy for a risk criterion. This was usually the ground motion experienced at a given return period, such as the peak ground acceleration or uniform probability acceleration response spectrum for a return period of 500 years. Recent developments have given much more emphasis to displacement and displacement response spectra.

2. EARTHQUAKE GROUND MOTION AND HAZARD IN AUSTRALIA

2.1 Introduction

The Australian mainland is itself a continent, and all of its earthquakes are within the Australia-India tectonic plate. This means that it has a relatively low rate of earthquake activity. However, its northern plate boundary with the Eurasian Plate is the very active region from Indonesia through Papua New Guinea, and its eastern boundary with the Pacific Plate is also a very active region from the Solomon Islands through Vanuatu, Fiji and Tonga to New Zealand. GPS data shows the Australian Plate moving very fast to the north northeast, at about 70 mm/year.

Tectonic stress measurements show that almost all of Australia is experiencing high level of horizontal compression and most faults show reverse faulting. A consequence of high compressive stress is that earthquakes can occur to very shallow depths. Shallow swarms of earthquakes often occur within one to three kilometres of the surface, with magnitudes ranging up to 4.0 to 5.0 respectively. Although heard loudly, and felt quite strongly, the high frequency motion has not usually produced significant damage beyond broken windows and cracked brickwork.

2.2 Specification of Earthquake Hazard

Earthquake hazard is quantified by ground motion recurrence, or the variation in a measure of ground motion (for example, peak ground acceleration, response spectra, or Arias intensity) with the recurrence interval (average return period, or its inverse the annual probability of exceedence). Ground motion recurrence can be obtained empirically from historical data in active regions, but the recurrence intervals for large earthquakes in Australia far exceeds the duration of both European settlement (just over 200 years) or aboriginal occupation (over 50,000 years).

The most common way of determining ground motion recurrence is the Cornell Method, or Probabilistic Seismic Hazard Assessment, PSHA, (Cornell, 1968; McGuire, 2004 for a detailed review). The Cornell method uses a number of models to define future earthquake activity, and to specify the ground motion resulting from this. The final stage is a multi-dimensional integration of ground motion probabilities covering all earthquakes likely to occur.

2.3 Earthquake Hazard in Adelaide

Adelaide experiences more earthquakes than any other Australian capital city. The most damaging was on 28 February 1954 when an earthquake of magnitude ML 5.4 occurred just south of the city causing widespread damage. Over 30,000 insurance claims were made, with a total value of tens of millions A$ (adjusted to 2012). Until the 1989 Newcastle ML 5.6 earthquake, Adelaide was the earthquake capital of Australia. Other larger but more distant earthquakes occurred in 1897 near Beachport to the southeast (Mw ~6.5), and in 1902 at Warooka to the west (Mw 6.0). For this study, a point at the geometric centre of the Central Business District has been used to evaluate hazard at Adelaide. This has often been done in the past, but without emphasis on displacement spectra.

2.4 Earthquake Recurrence Model

The earthquake recurrence model used for this study is derived from the model AUS5 (Brown and Gibson, 2004). This includes source area zones based on geological and geophysical data, and quantified using historical seismicity. The model also includes active faults in the Adelaide region, delineated by geology, and quantified using estimated fault slip rates. The earthquakes attributed to the active faults have been subtracted from the earthquake activity within the area zones in which they are located. Figure 1 shows the areas around Adelaide with historical earthquake activity, the chosen area source zones, and active faults in black, especially to the east of Adelaide.

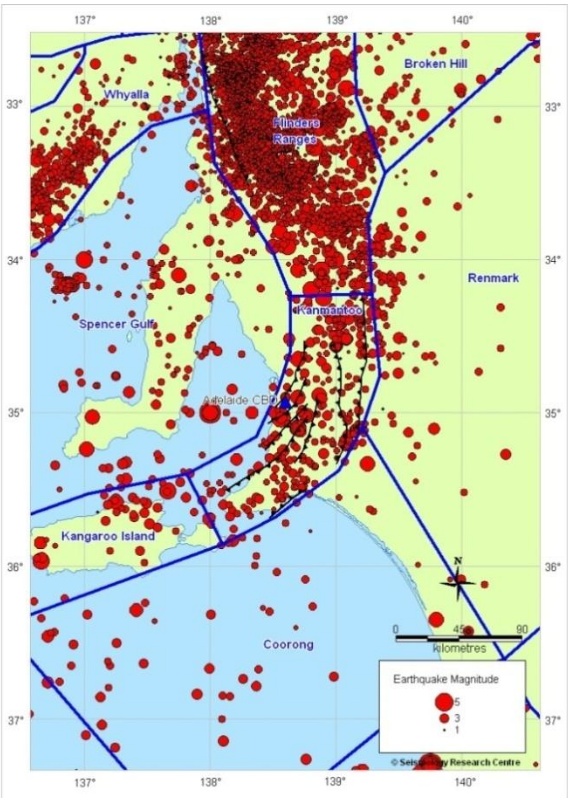

Figure 1. Earthquake recurrence model AUS5 in the Adelaide region

2.5 Ground Motion Model

There is insufficient strong motion data to allow development of a ground motion model specifically for the Adelaide area. A range of ground motion models determined using data from elsewhere were used to select the most relevant for Adelaide. The main problem involved with displacement response is that it inherently emphasises long period motion. Strong motion data recorded in the past using analogue accelerographs gave no meaningful displacement results. Early digital accelerographs performed better, but not at longer periods beyond about four seconds, where accelerations disappeared into instrument noise levels. High dynamic range strong-motion accelerographs perform better again, but they a relatively new, have recorded few large nearby earthquakes, and some of this data is not yet freely available. The only ground motion models developed using Australian data that is currently available are those by Somerville and others (2009). These include a Yilgarn Cratonic model applicable for the Yilgarn Shield east of Perth, and a Non-Cratonic model applicable in Eastern Australia. Although there are Proterozoic rocks in the Adelaide region, these are mainly sedimentary and much younger than the Archean Cratonic rocks of the Yilgarn. The Somerville (2009) Cratonic model seems a logical choice.

Most of the previous response spectral models were developed to give acceleration response only, and these do not provide any useful displacement response results. The Next Generation Attenuation ground motion models included many improvements, one of which was to improve displacement motion estimates. Four NGA ground motion models have been published (Power and others, 2008), all using data mainly from western USA, but with some relevant data from other locations. One of the first observations was that they tend to be more consistent with each other, especially at longer periods where magnitudes are defined, but they vary at short periods (with stronger peak ground accelerations when applied in regions of high stress, and lower accelerations in low stress regions). Although not applicable for the geological extremes of cratonic regions or for actively subducting plate boundaries, they seem to be applicable to many regions with Palaeozoic continental crust.

The Boore and Atkinson (2008) NGA function was for lower stress earthquakes, so not considered appropriate for the Adelaide region. Results for the other three NGA functions by Abrahamson and Silva (2008), Campbell and Bozorgnia (2008) and Chiou and Youngs (2008) have been computed. The four ground motion models have each been applied to all relevant source zones and active faults, so results can be compared. The resulting acceleration and displacement spectra for the 475 year and 2475 year return period level hazard on deep and/or soft soil sites (V_{s30} = 257 m/s) are given in Figures 2 and 3.

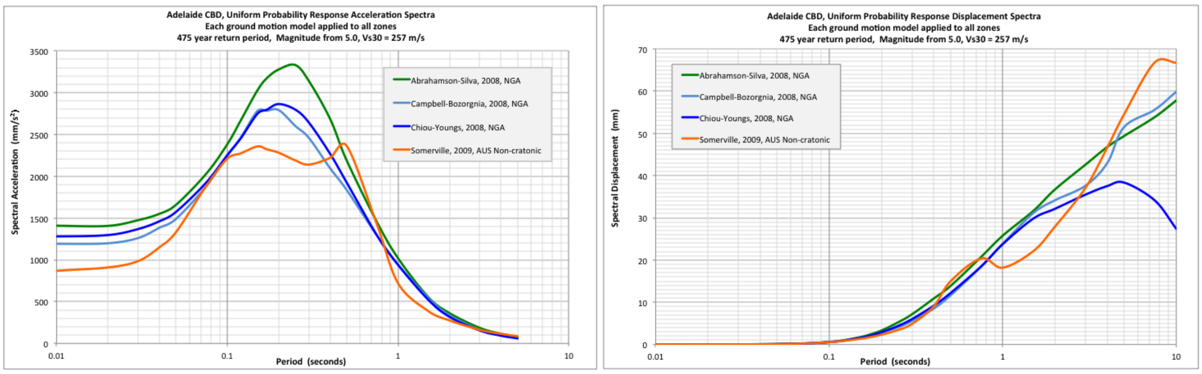

Figure 2. Adelaide, Magnitude 5.0 plus, 475 year, Vs30 = 257 m/s, Acceleration and Displacement Spectra

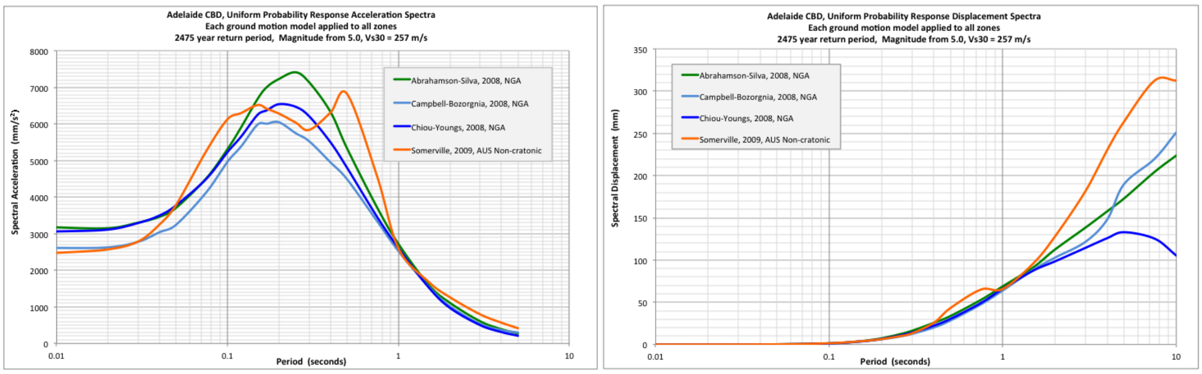

Figure 3. Adelaide, Magnitude 5.0 plus, 2475 year, Vs30 = 257 m/s, Acceleration and Displacement Spectra

3. WALL CASE STUDY

3.1 Parameters

A preliminary study has been conducted to determine the approximate displacement capacity of reinforced concrete walls in low to medium rise buildings with dimensions, reinforcement ratios, axial load ratios, material properties and detailing that are representative of the walls that are designed using the current Australian Standard for concrete structures, AS3600 (Standards Australia, 2009). It uses an approximate displacement-based analysis procedure that is intended to identify situations where there may be a problem. This study is based on walls that are typical of those that are built around stairs and lifts in the core of reinforced concrete buildings in Australia, with 2 walls in each directions, wall lengths of 7m and 3 m, and a wall thickness of 200 mm (150 mm is sometimes used). Buildings of 5, 10 and 15 stories are considered and the assumed height of each storey is 3.2 metres. In terms of aspect ratio this corresponds to aspect ratios of 2.3, 4.6 and 6.9 for the 7 metres wall, and 5.3, 10.7 and 16 for the 3 metre wall.

The ratio of the cross-sectional area of the reinforcing steel to the concrete has a specified minimum in AS3600 (Standards Australia, 2009) of 0.0015 in the vertical direction and 0.0025 in the horizontal direction. The maximum centre-to-centre spacing of parallel bars is designated as the lesser of 2.5 times the wall thickness and 350 mm. Another requirement in the Standard is that vertical and horizontal reinforcement should be provided in two grids, one near each face of the wall if the wall is greater than 200 mm thick. It is common practice to provide two grids for a 200 mm wall. The vertical reinforcement ratios (p_{wv}) considered in this study are as follows: 0.0016 (1 N12@350 mm), 0.0032 (2N12@350), 0.0057 (2N12@200), 0.0155 (2N20@200), 0.03 (2N24@150). Only in one of these cases, N12@350mm, is a single grid of reinforcing used in the middle of the wall. In the horizontal direction, it is common to provide 2N12@200 mm, but it is possible to go as low as 2N12@350.

The Australian earthquake loading standard (AS1170.4) requires that a gravity load of G+0.3Q (permanent load +0.3 times imposed load) is considered to act simultaneously with the design level earthquake loading. This commonly results in axial load ratios (ALRs) in the walls that range from 0.05 to 0.3. The seismic mass (M_s) that needs to be catered for by a wall typically varies from 100 tonnes per floor to 600 tonnes per floor; values of 100, 300 and 600 have been included in this study.

3.2 Material properties

The reinforcing steel is designated N type in AS3600 ("N" for normal ductility) and is required to have a minimum characteristic value of yield strength (f_y) of 500 MPa, ratio of ultimate strength to yield strength (f_t/f_y) of 1.08, and of uniform elongation strain (ε_u) of 0.05 (5%). Mesh steel is sometimes used in walls; it is "L" type, with a low ductility; the specified minimum characteristic elongation strain is just 0.015 (1.5%). The concrete in the walls is assumed to have a characteristic compressive strength (f_c') of 40 MPa (values of 32 MPa and 40 MPa are common in low to medium rise construction). The detailing is such that very little or no confinement of concrete would be activated.

3.3 Wall displacement capacity

The displacement capacity is found using a similar approach to that outlined in (Priestley et al, 2007). There are some important differences, mainly because in this study there is lack of confinement in the concrete; also consideration is given to axial load ratios higher than 0.015 which is the maximum value considered in (Priestley et al, 2007), and higher values of seismic mass per floor per wall are also included in this study.

3.4 Moment vs. Curvature

A series of analyses have been conducted to determine key points on the moment vs curvature graph for the base of walls which have parameters with the ranges given above. The key points include the moment and curvature when cracking is initiated, at first yield of the steel, and when the strain in the concrete reaches 0.003 in compression (the ultimate condition in AS3600). The variations in yield curvature (ε_y) and yield moment (M_y) with ALR and p_{wv} are given in Figure 4 for the 3 m and 7 m long walls. There is only a slight variation in the yield curvature with reinforcement ratio. The maximum curvature in the section at the base of the wall is determined by placing limits on the compressive strain in the concrete and the tensile strain in the steel. For some performance limits it may also be limited by the maximum drift that is allowed in the wall (worst case will be in the top storey). In this study, following recommendations in (Priestley et al, 2007) for a 2475 year return period event and a collapse prevention performance level, the tensile strain in the steel is limited to $0.9\varepsilon_u$ (= 0.045 for N type steel) and the compressive strain in the concrete is limited to 1.5 times 0.003, i.e. 0.0045. There is no limit placed on the drift (although some consideration of drifts will be included later). Because of the lack of confinement of the concrete, it is the compressive strain in the concrete that usually governs.

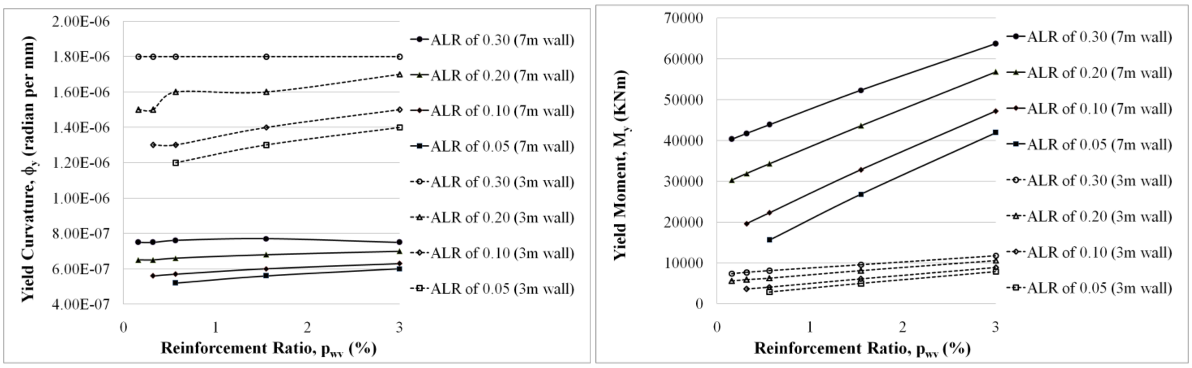

Figure 4. Variation of yield curvature and moment with ALR and p_{wv} for the 3 m and 7 m long walls

3.5 Load vs. Displacement

The walls are to be analysed as single degree of freedom substitute structures with an effective mass, m_e, and effective stiffness, K_e. Following recommendations in (Priestley et al, 2007) a linear curvature distribution has been assumed when calculating both the displacement at cracking, Δ_{cre}, and the displacement at yield, Δ_{ye}, at the effective height of the substitute structure, H_e. Assuming that the wall is able to form a plastic hinge at the base without having undergone a premature failure (such as a shear failure), the additional displacement that can occur at the effective height is calculated based on a simple plastic mechanism, with the wall rotating as a rigid body about the base. The plastic rotation at the base is determined by the difference between the maximum curvature and the yield curvature multiplied by the plastic hinge length. The plastic hinge length has been calculated using the same approach as in (Priestley et al, 2007) with adjustments made due to the use of N type steel rather than the higher ductility steel that is usually used in regions of high seismicity.

An example of the load vs. displacement graphs for single degree of freedom substitute structures representing 7m long walls with the same reinforcement ratio (p_{wv} = 0.57%) and axial load ratio (ALR = 0.1) at the base, and in buildings with 5 storeys (Aspect ratio = Height to length = H_n to l_w = 2.3), 10 storeys (Aspect ratio = 4.6) and 15 storeys (Aspect ratio = 6.9) is given in Figure 5. The graphs indicate where cracking (cr), yielding (y), ultimate (u) and maximum displacement (max) have occurred. In the graph for the 10-storey building, points are defined to show the displacement and force in the substitute structures at cracking (Δ_{cre}, P_{cre}), yield (Δ_{ye}, P_{ye}), ultimate (Δ_{ue}, P_{ue}) and the maximum displacement capacity (Δ_{maxe}, P_{maxe}). The secant stiffness at initiation of flexural cracking, yield and at the maximum displacement are given as K_{cre}, K_{ye} and K_{maxe} respectively.

Figure 6 illustrates the relative contribution from the elastic (up to yield) and plastic displacements for walls with varying aspect ratios and axial load ratios. The walls are 7m long with reinforcement ratios of 0.57% and axial load ratios of 0.1 at the base. The contribution of the plastic displacements is low for the higher axial load ratios as expected.

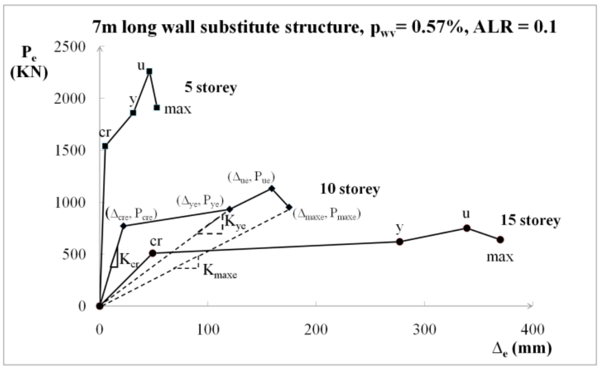

Figure 5. Example of Load vs. Displacement Graphs

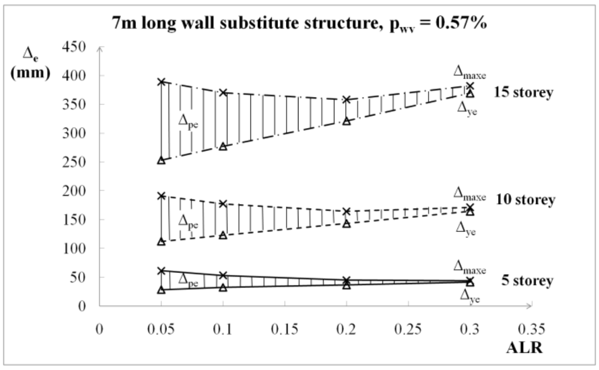

Figure 6. Relative contribution of elastic and plastic displacements for different ALRs

3.6 Calculation of wall periods

The periods of the substitute structures representing the walls have been calculated at cracking, at yield and at maximum displacement using the effective (secant) stiffnesses that have been defined in Figure 5: K_{cre}, K_{ye} and K_{maxe} respectively. The effective mass of the SDOF substitute structures is taken as 0.75 times the total seismic mass acting on the wall. The effective periods at cracking and at yield for 3m and 7m long walls with axial load ratios of 0.1 and a vertical reinforcement ratio of 0.57% are given in Figure 7. For the 7m wall with a reinforcement ratio of 0.57% the difference between the period at yield and at maximum displacement decreases as the axial load ratio increases. This is shown in Figure 8 and is a valid trend for R/C walls in general.

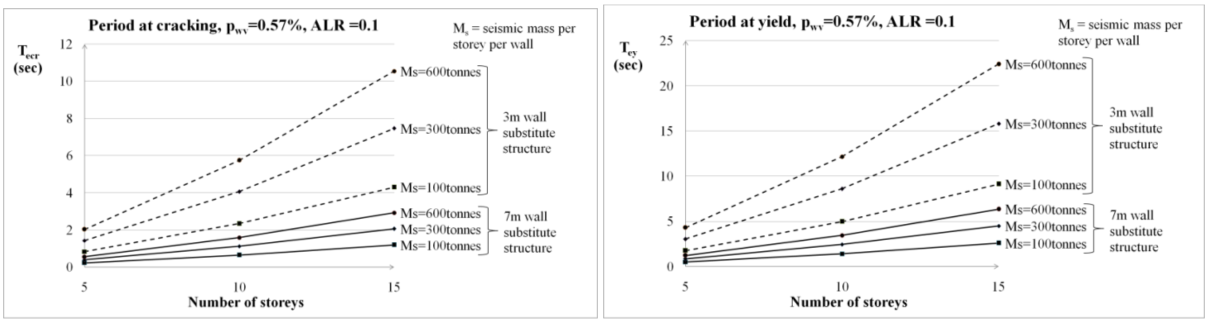

Figure 7. Period at cracking and at yield for the 3m and 7 m walls with different seismic masses

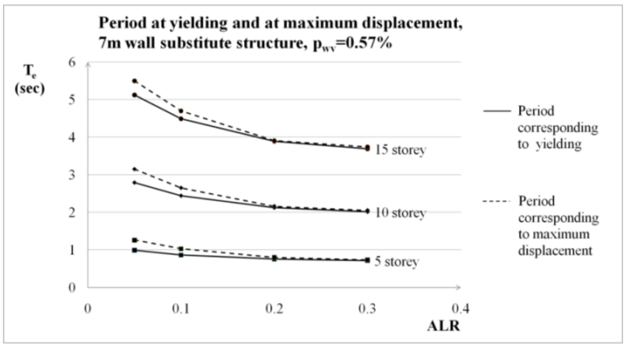

Figure 8. Variation of the period at yielding and maximum displacement with ALR

4. ASSESSMENT OF THE WALLS

The worst case site conditions (V_S30 = 257 m/sec, deep and/or soft soil) have been assumed when assessing the walls. The assessment has been carried out for both the 475 year return period and the

2475 year return period seismic hazard. In the results presented here, the Somerville, non-cratonic model has been used in the estimation of displacement demands. The walls have first been assessed by determining if they have yielded. The periods at yield have been used in this assessment. If the walls have yielded then a further assessment has been carried out using the period at maximum displacement to determine the displacement demands, and an assessment has been made as to whether the walls have sufficient displacement capacity. Table 1 and Table 2 show the results of these assessments for the 5 storey walls of 7m length. Results for the 10-storey walls of 7 m length and the 3 m walls are not presented here due to lack of space. The wall assessment has been based on the displacement demands found from the elastic displacement spectra with damping of 5%, even for those walls that have yielded. This conservative approach is based on the unknown potential of the walls to undergo a buckling failure of the longitudinal bars or an out-of-plane failure. However, the reduced displacement demands that include the effect of increased damping determined using the methods given in (Priestley et al, 2007) are given in brackets in Table 2 for the walls that have yielded. Please note that in Table 1, given that none of the walls had yielded for an M_s value of 600 tonnes and a return period of 475 years, it is obvious that they would not have yielded for the M_s values of 100 or 300 tonnes; hence the displacement demands for these M_s values have not been presented. Similarly in Table 1 it is clear that the walls have all yielded for an M_s value of 300 tonnes at a return period of 2475 years, so it is clear that they would yield for the M_s value of 600 tonnes (since the periods are higher); hence the displacement demands are not given at this M_s value .

Table 1. Assessment of 7m long 5-storey walls - Assumed wall state: Cracked but not yielded

		Seismic mass (M_s)/floor/wall (tonnes)							Results			
		100		300		600			475yrRP	2475yrRP		
ALR	p_{wv}	T_{ye}	δ_d	T_{ye}	δ_d	T_{ye}	δ_d	Δ_{ye}	Wall exceeded yield?			
		(sec)	2475yr RP	(sec)	2475yr RP	(sec)	475yr RP		M_s/floor/wall (tonnes)			
	(%)		(mm)		(mm)		(mm)	(mm)	ALL	100	300	600
0.05	0.565	0.6	52	1.0	66	1.4	23	28	N	Y	Y	Y
	1.55	0.5	43	0.8	66	1.1	22	30	N	Y	Y	Y
	3	0.4	26	0.6	52	0.9	22	32	N	N	Y	Y
0.1	0.32	0.5	43	0.9	66	1.3	22	30	N	Y	Y	Y
	0.565	0.5	43	0.9	66	1.2	22	31	N	Y	Y	Y
	1.55	0.4	26	0.7	60	1.0	22	32	N	N	Y	Y
	3	0.4	26	0.6	52	0.9	22	34	N	N	Y	Y
0.2	0.16	0.5	43	0.8	66	1.1	22	35	N	Y	Y	Y
	0.32	0.4	26	0.8	66	1.1	22	35	N	N	Y	Y
	0.565	0.4	26	0.7	60	1.1	22	36	N	N	Y	Y
	1.55	0.4	26	0.7	60	1.0	22	37	N	N	Y	Y
	3	0.3	14	0.6	52	0.8	22	38	N	N	Y	Y
0.3	0.16	0.4	26	0.7	60	1.0	22	41	N	N	Y	Y
	0.32	0.4	26	0.7	60	1.0	22	41	N	N	Y	Y
	0.565	0.4	26	0.7	60	1.0	22	41	N	N	Y	Y
	1.55	0.4	26	0.7	60	0.9	22	42	N	N	Y	Y
	3	0.3	14	0.6	52	0.8	22	41	N	N	Y	Y

Under the 475 year return period earthquake, most of the 7m walls considered in this study did not yield and had sufficient displacement capacity. The exceptions to this are the following walls with low vertical reinforcement ratios and low axial load ratios: ALR of 0.05 and reinforcement ratios of 0.16 and 0.32, and ALR of 0.1 and a reinforcement ratio of 0.16. In these cases the ultimate moment capacity is less than 1.2 times the moment at cracking. There is a risk of very sudden failure in these walls, especially those for which the cracking moment is greater than the ultimate moment capacity. The wall would crack and then fail in a brittle manner with a high concentration of strain in the reinforcing steel at the crack and a strong likelihood of steel fracture. The displacement demands at the periods corresponding to the uncracked stiffness of the wall have been found for all of these walls from the displacement spectrum for a 475 year return period. The demands are greater than the cracking displacement of the wall for all of the 5 and 10 storey walls except that for the 5 storey wall with M_s = 100 tonnes. The cracking displacement capacity is greater than the displacement demand for the 15 storey walls. However, given the brittle nature of this type of failure, walls with these properties have not been considered in the remaining assessment.

Table 2. Assessment of 7m long 5-storey Walls - Assumed wall state: at maximum displacement

		Seismic mass (M_s)/floor/wall (tonnes)								Results		
		100		300		600				2475yrRP		
ALR	p_{wv}	T_{maxe}	δ_d	T_{maxe}	δ_d	T_{maxe}	δ_d	Δ_{maxe}	Δ_{ye}	Disp. capacity sufficient?		
		(sec)	2475yr RP	(sec)	2475yr RP	(sec)	2475yr RP			M_s/floor/wall (tonnes)		
	(%)		(mm)		(mm)		(mm)	(mm)	(mm)	100	300	600
0.05	0.565	0.7	60(45)	1.3	88(61)	1.8	122(80)	61	28	Y	N (Y)	N
	1.55	0.5	43(37)	0.9	66(50)	1.2	80(59)	52	30	Y	N(Y)	N
	3	N/A	N/A	0.7	60(48)	1.0	66(50)	48	32	N/A	N(Y)	N
0.1	0.32	0.7	60(46)	1.1	73(54)	1.6	110(75)	55	30	N(Y)	N (Y)	N
	0.565	0.6	52(42)	1.0	88(63)	1.5	96(67)	53	31	Y	N	N
	1.55	N/A	N/A	0.8	66(51)	1.1	73(55)	48	32	N/A	N	N
	3	N/A	N/A	0.6	52(44)	0.9	66(51)	46	34	N/A	N (Y)	N
0.2	0.16	0.5	43(39)	0.9	66(52)	1.2	80(60)	46	35	Y	N	N
	0.32	N/A	N/A	0.8	66(52)	1.2	80(60)	46	35	N/A	N	N
	0.565	N/A	N/A	0.8	66(52)	1.1	73(57)	45	36	N/A	N	N
	1.55	N/A	N/A	0.7	60(49)	1.0	66(52)	46	37	N/A	N	N
	3	N/A	N/A	0.6	52(45)	0.9	66(53)	48	38	N/A	N (Y)	N
0.3	0.16	N/A	N/A	0.8	66(54)	1.1	73(62)	44	41	N/A	N	N
	0.32	N/A	N/A	0.7	60(51)	1.1	73(63)	44	41	N/A	N	N
	0.565	N/A	N/A	0.7	60(51)	1.0	66(54)	44	41	N/A	N	N
	1.55	N/A	N/A	0.7	60(51)	1.0	66(55)	46	42	N/A	N	N
	3	N/A	N/A	0.6	52(47)	0.9	66(54)	45	41	N/A	N	N

For the 5 storey walls subject to the 2475 year return period hazard, many of the walls with an M_s of 100 tonnes do not yield (see Table 1). All of the walls with seismic mass per floor per wall (M_s) of 300 tonnes or 600 tonnes do not have adequate displacement capacity (see Table 2). All of the 5-storey walls that yield have been shown to have adequate displacement capacity except for the wall with an ALR of 0.1 and a reinforcement ratio of 0.32%. For the 10 storey walls subject to the 2475 year return period hazard, most of those with M_s values of 100 and 300 tonnes do not reach yield. Those that do reach yield have adequate displacement capacity. However, many of the 10 storey walls with M_s = 600 tonnes do not have adequate displacement capacity. Only two of these do not reach yield; those with an ALR of 0.3 and p of 1.55% and 3.0%. Of the remaining walls with an M_s of 600 tonnes, only those with a reinforcement ratio of 3% (ALRs of 0.05, 0.1 and 0.2) have adequate displacement capacity, since the higher effective stiffness due to the high reinforcement ratio results in lower displacement demands. For the 15 storey walls subject to the 2475 year return period hazard, most of the walls do not yield, and those that do have sufficient displacement capacity.

The possibility of premature shear failure in the walls has also been investigated. The shear force is based on the shear force at yield multiplied by the ratio of the displacement demand to the displacement at yield for those walls that have not yielded. For walls that have yielded it is based on the shear in the wall at ultimate conditions. The shear capacity has been assessed using the appropriate formula in Section 11 of AS3600. The "steel" component of this capacity is directly proportional to the horizontal reinforcement ratio in the wall. For the 5-storey walls 2N12s at 350 mm spacing is generally adequate, although 2N20s at 200 mm spacing is sometimes required. The 10-storey and 15-

storey walls have also been investigated and 2N12s at 350 mm spacing are adequate for all of these. Also, it is important to take a holistic approach to the assessment of the building. Even though the walls may be able to tolerate high levels of drift in the upper floors, it is unlikely that the non-ductile frames that are present to take gravity loads will. An initial assessment has been made to determine the drifts at the top storey of the walls. For the 7 m walls, if the drift limit is considered to be 1.5%, then the 5 and 10 storey walls would satisfy this criteria. However, many of the 15-storey walls would exceed this limit and a further investigation of the secondary framing system would need to be made. Another consideration here is that there may be accidental torsional effects that would increase the drifts experienced by the secondary framing, especially if the core were placed eccentrically.

5. CONCLUSION

A seismic structural assessment of non-ductile reinforced concrete walls that have parameters that are commonly used in Australia has been conducted. The assessment uses the latest seismic hazard techniques available in Australia. Displacement response spectra generated using these techniques are then incorporated in a displacement-based approach to the structural assessment of the walls. Results have been presented for walls of 7m length and 200 mm thickness, with axial load ratios varying from 0.05 to 0.3, vertical reinforcement ratios varying from 0.16% to 3 %, and wall heights of 16,32 and 48 metres. The displacement response due to the 475 year return period earthquake hazard does not cause yield of any of the walls and hence little damage would be expected under this level of hazard. However, in the 2475 year return period event many of the walls are shown to be inadequate. Given the poor performance of similar non-ductile reinforced concrete structures in Christchurch when subjected to a 2475 year return period event for that city, a reassessment needs to be made of the risks associated with not requiring adequate performance under this level of event in the Australian Standards.

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