Seismic Assessment of the RC building stock of Melbourne from rare and very rare earthquake events

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ABSTRACT:

This paper focuses on a seismic vulnerability assessment of the Australian reinforced concrete structural (or shear) wall building stock using the Melbourne CBD as a case study. Each of the 1403 reinforced concrete buildings used in the assessment are assumed to be laterally supported by rectangular (peripheral) or C-shaped core walls. The assessment was conducted based on the Capacity Spectrum method, which involves a comparison between the capacity and demand curves in the form of acceleration-displacement response spectra. Plastic hinge analysis expressions were used to derive the capacity curves of the buildings. The 500-year ("rare") and 2500-year ("very rare") return period spectra derived from a Probabilistic Seismic Hazard Analysis for the city of Melbourne using the AUS5 earthquake recurrence model were used for the earthquake demand. The Australian Seismic Site Conditions Map from Geoscience Australia is used to find the expected site conditions for each building, while SHAKE-2000 is used to amplify the expected earthquake demand. The results show that only 34 (2.4%) reinforced concrete buildings used in this assessment were deemed to reach the Collapse Prevention performance level for the 500-year return period event, whereas a total of 540 (38.5%) buildings were estimated to reach this performance level for the 2500-year return period event.

Keywords: vulnerability, shear walls, collapse, damage distribution

1. Introduction

The aim of this research is to assess the seismic performance of typical reinforced concrete (RC) structural (or shear) wall buildings in Australia when subjected to a rare or very rare earthquake event, corresponding in this research to 500-year and 2500-year return periods respectively. The city of Melbourne in Victoria is used as a case study for this research. For seismic vulnerability studies, a good prediction of both *seismic demand* and *structural capacity* is required. Therefore, the seismic demand in Melbourne needs to be assessed in order to derive the acceleration-displacement response spectra (ADRS) for different return period intervals. Moreover, plastic hinge analysis expressions that can accurately predict the displacement capacity of lightly reinforced and unconfined RC walls are required to derive the capacity curves. Using the Capacity Spectrum method for a large number of different buildings, the RC structures in Melbourne can be assessed for "rare" or "very rare" earthquake events.

2. Seismic demand

The probability of a particular earthquake intensity being exceeded for a given location and time recurrence can be derived from a Probabilistic Seismic Hazard Analysis (PSHA) (Cornell, 1968). A typical output of a PSHA is the estimated acceleration (and displacement) response spectra for a range of return periods (or annual frequency of exceedance).

The AUS5 earthquake recurrence model, developed by Brown and Gibson (2004), assumes a relationship between the current seismicity and the geology together with the past and present tectonics. Many earthquake recurrence models assume uniform seismicity over large areas, and they tend to give a much lower hazard for "active" regions compared with models that are based on known seismicity. In contrast, earthquake recurrence models that assume that the future seismic activity will only occur where known past earthquakes have occurred will give a wide range of hazard in different locations. The AUS5 model, with 'source zones of dimensions tens to hundreds of kilometres' and with smoothed seismicity, lies in between the two extremes (Gibson & Dimas, 2009). A maximum moment magnitude (M_w) of 7.5 is adopted for this study given the recommendations in Burbidge (2012) and Clark *et al*. (2011), while a minimum M_w of 5.0 was used. In areas where faults are incorporated, the activity generated by the faults are subtracted from the total activity in the area of the respective zone. In other words, a 'subtraction method' was used for accumulating the fault and area seismicity by using a background source if there are faults within a specific zone (Dimas *et al.*, 2016).

EZ-FRISK (McGuire, 1995) is a frequently used computer program in seismology and earthquake engineering for carrying out PSHA studies. The Cornell (1968) method is utilised in calculating the seismic hazard, while the program also relies on calculations from Youngs and Coppersmith (1985) for the estimation and conversions of seismic activity from slip rates of active faults. Each fault is assigned a slip rate in metres per million years, which 'either refers to the vertical offset rate or the horizontal offset rate' (Dimas *et al.*, 2016).

The selection of Ground Motion Prediction Equations (GMPEs) representing the attenuation of the seismic waves for the different regions in Australia is a critical factor in the determination of the resulting seismic hazard. The dataset of strong-motion earthquake events in Australia is insufficient to derive an accurate GMPE for the Australian conditions. It is more appropriate to adopt an attenuation function that has been well developed for a region that has similar geological conditions to the region that is being investigated. Therefore, the NGA-West 1 function from Chiou and Youngs (2008) was used as the GMPE for "Non-Cratonic" Australia; this was due to previous research indicating that the function 'generally fit in between the two extremes' of the two attenuation functions that were derived specifically for Australia (Hoult *et al.*, 2013). Moreover, the function had also been used in past PSHA studies to represent the attenuation in the "Non-Cratonic" areas of Australia (Burbidge, 2012; Gibson & Dimas, 2009; Goldsworthy & Gibson, 2012).

The seismic hazard has been calculated using EZ-FRISK for the city of Melbourne in Australia, using the same latitude and longitudinal coordinates (144.96, -37.81) that were used to derive the hazard values from the 2012 Australian Earthquake Hazard Map (Leonard *et al.*, 2013).

The seismic hazard results from the PSHA obtained using the AUS5 earthquake recurrence model is given in Table 1 as the *PGA* on site class B_e (V_{s30} = 760m/s) for the two return periods used in this study. For comparisons, the *PGA* values for Melbourne, and for the two return periods, from the AS 1170.4:2007 (Standards Australia, 2007), GA (Leonard *et al.*, 2013) and the proposed values from GA for 2018 are also given in Table 1. The *PGA* values from this study, using the AUS5 earthquake recurrence model, give a larger prediction of "hazard" for the city of Melbourne and for the two return periods in comparison to others. However, the *PGA* can be a poor indicator of the seismic demand and it is therefore important to compare the resulting acceleration and displacement spectra from the different sources. Figure 1 gives the resulting acceleration and displacement response spectra for 500-year and 2500-year return periods for Melbourne using the AUS5 model. Superimposed in these figures are the derived spectra from AS 1170.4:2007 for the two return periods. Moreover, the acceleration spectra from Leonard *et al*. (2013) for Melbourne has been superimposed on Figure 1(a) for the range of spectral period that was available (0.01s to 1.00s).

Table 1 PGA seismic hazard values for Melbourne

Return Period (Years)	AUS5	AS 1170.4:2007	GA (2013)	GA (2018)
500	0.110	0.080	0.059	0.055
2500	0.250	0.144	0.157	0.150

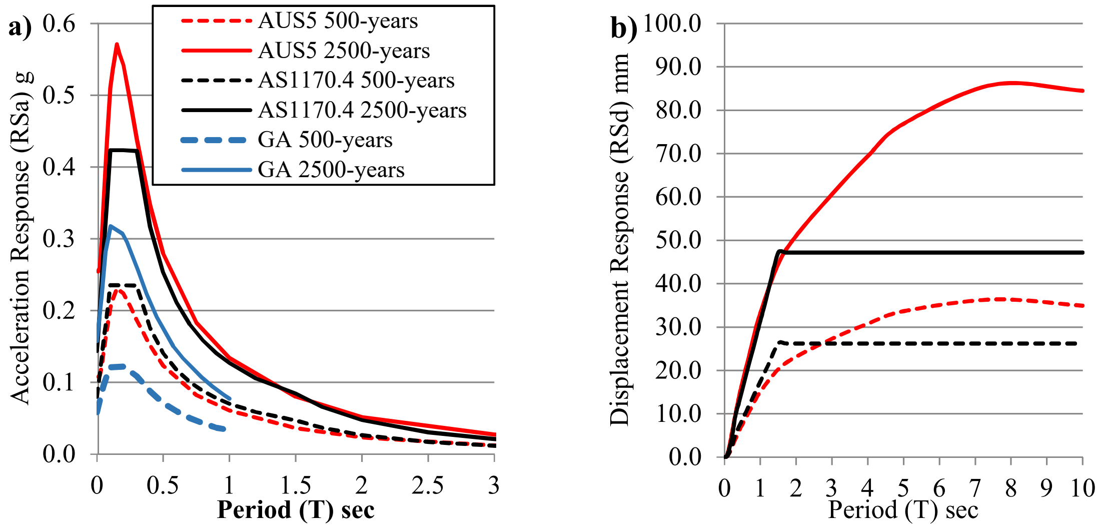

Figure 1 (a) acceleration and (b) displacement response spectra for Melbourne

The resulting acceleration and displacement response spectra from the PSHA study using the AUS5 earthquake recurrence model will be used to assess the RC structural wall buildings in the event of 500-year and 2500-year return period earthquake events in Melbourne.

3. Site response

Identification of the site class at the location of the buildings is necessary to amplify (or, deamplify) the ground motions to provide an accurate response spectrum of the site used in assessing vulnerability. Geoscience Australia conducted a study to provide a National Regolith Site Classification Map (McPherson & Hall, 2007). This was recognised as being an important tool for modelling earthquake events, where the map could provide information on the 'potential influence of variation in geological materials on the ground shaking' (McPherson & Hall, 2007). More recently, a revised Seismic Site Conditions (SSC) map for Australia (McPherson, 2017) was completed by GA that integrated 'new and revised geological data published since 2007'. The maps from GA use soil classifications that were defined by the shear wave velocity of the top 30 m below the surface (V_{s30}), similar to the current classification of some soils in AS 1170.4:2007. The resulting map from McPherson (2017) for Melbourne is illustrated in Figure 2(a) with the different coloured regions corresponding to the different soil classes. The SSC map from McPherson (2017) uses seven site classes that are based on the modified NEHRP site classifications, modified by Wills *et al.* (2000) to suit the Australian conditions. This information can be used to estimate the site response for different ground motions using an equivalent linear analysis and shear wave velocity profiles corresponding to the modified NEHRP classes.

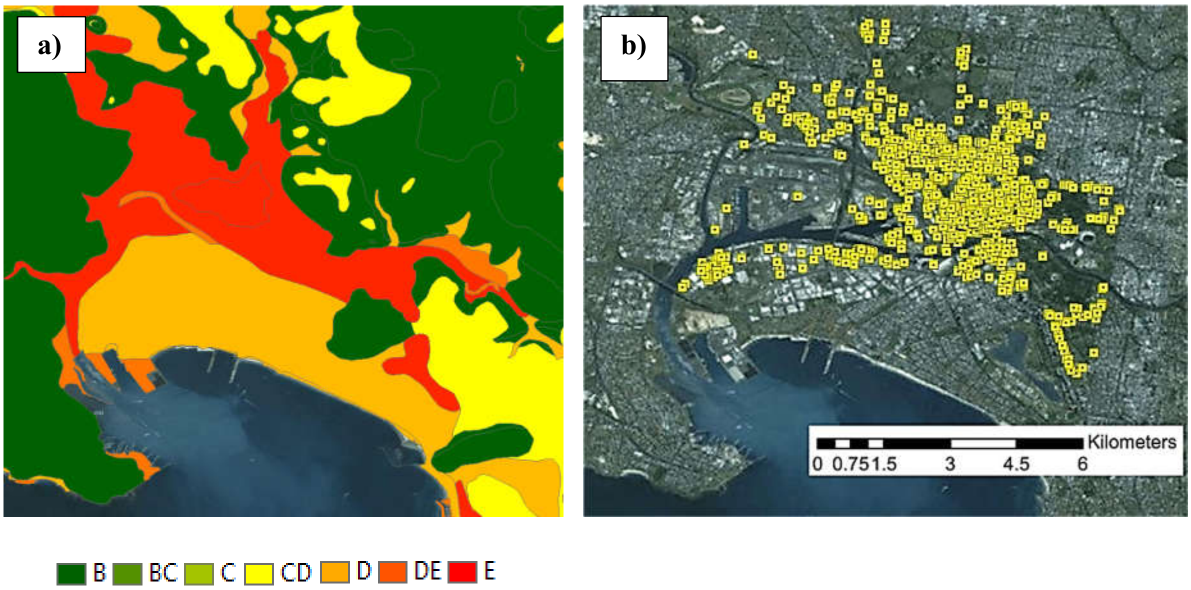

Figure 2 (a) Seismic site conditions map of Melbourne from McPherson (2017) and (b) locations of RC buildings from the CLUE dataset (Melbourne City Council, 2015)

Seven acceleration time-histories were generated from SeismoArtif (SeismoSoft, 2013) for both the 500-year and 2500-year return period events, where the corresponding acceleration response spectra were used as targets. An intraplate setting was used in SeismoArtif to generate the artificial ground motion, while magnitude-distance (M-R) combinations of M6R30 and M6R11 were subsequently used for the 500-year and 2500-year return period spectra respectively. The distance from the Melbourne CBD to the Beaumaris and Yarra faults were used for appropriate values of *R*, while an approximate value for *M* was based on the CAM model (Lam *et al.*, 2003; Lam *et al.*, 2000). The ground motions from these M-R combinations are ultimately scaled within the SeismoArtif program such that the resulting acceleration response spectra fits within +/- 10% of the target spectrum. An example of one of the resulting acceleration time-histories is given in Figure 3(a) for both the 500-year and 2500-year return period events, while the resulting (mean) acceleration response spectra from these artificial ground motions are shown in Figure 3(b) superimposed on the target spectra.

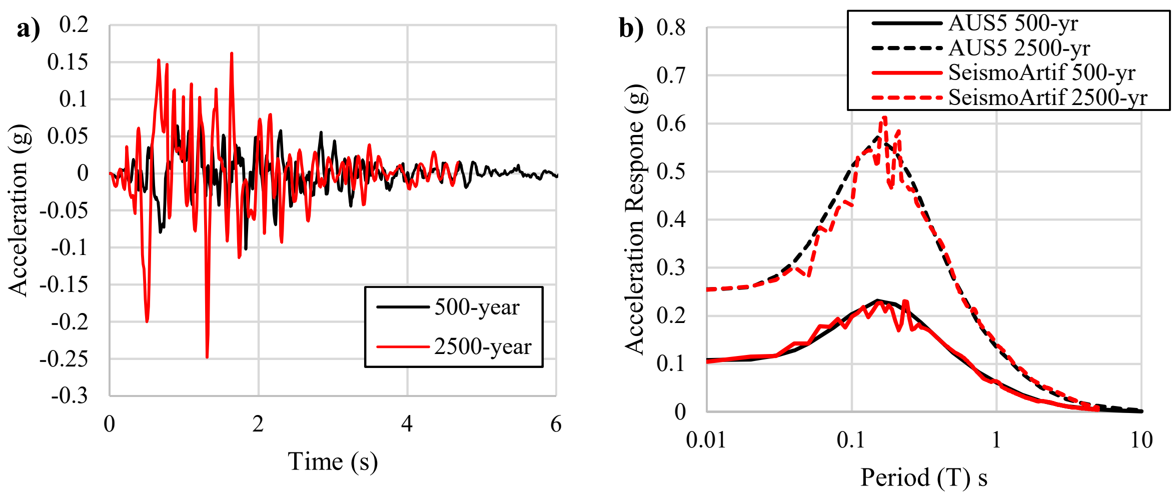

Figure 3 (a) acceleration time-histories and (b) acceleration response spectra

The generated artificial acceleration time-histories were used in SHAKE2000 (Ordonez, 2013) for equivalent linear analyses using the different shear wave velocity profiles corresponding to the different modified NEHRP soil classes. The shear wave velocity profiles were provided from the research by Roberts *et al.* (2004) and Kayen *et al.* (2015), which were predominantly taken from sites in the Melbourne region. For site classes *BC*, *C* and *CD*, a clay material was assumed for layers with a V_s less than 760 m/s, which is the lower limit considered by some for "soft rock" (Building Seismic Safety Council, 2009). A sand material was used for the entire profile for site classes *D*, *DE* and *E*, as these soil classes are commonly attributed to deep alluvial sites for Melbourne (McPherson & Hall, 2007). More information on the material models chosen and the setup used in SHAKE2000 can be found in Hoult *et al.* (2016). The resulting (mean) acceleration and displacement response spectra for the different soil classes are given in Figure 4 and Figure 5 for the 500-year and 2500-year return period respectively.

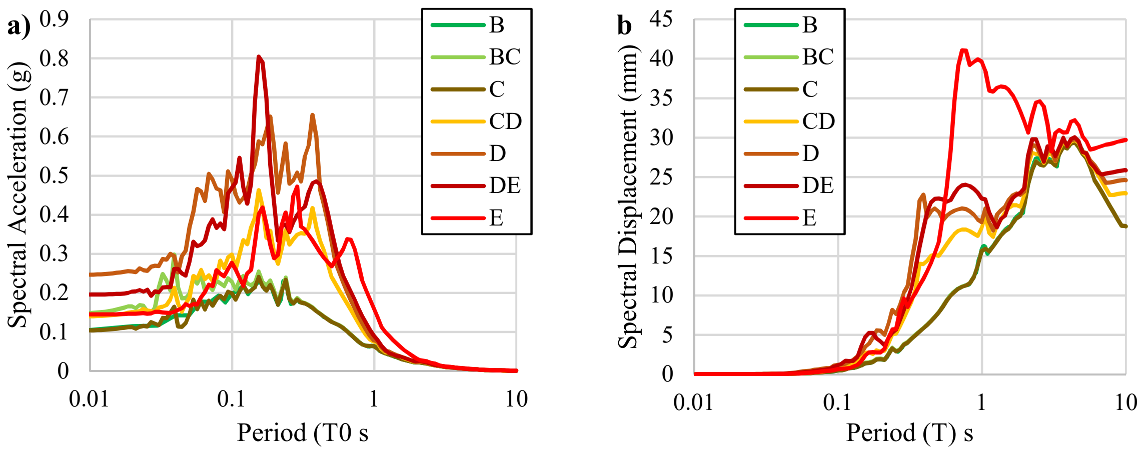

Figure 4 500-year return period spectra (a) acceleration response and (b) displacement response acceleration response

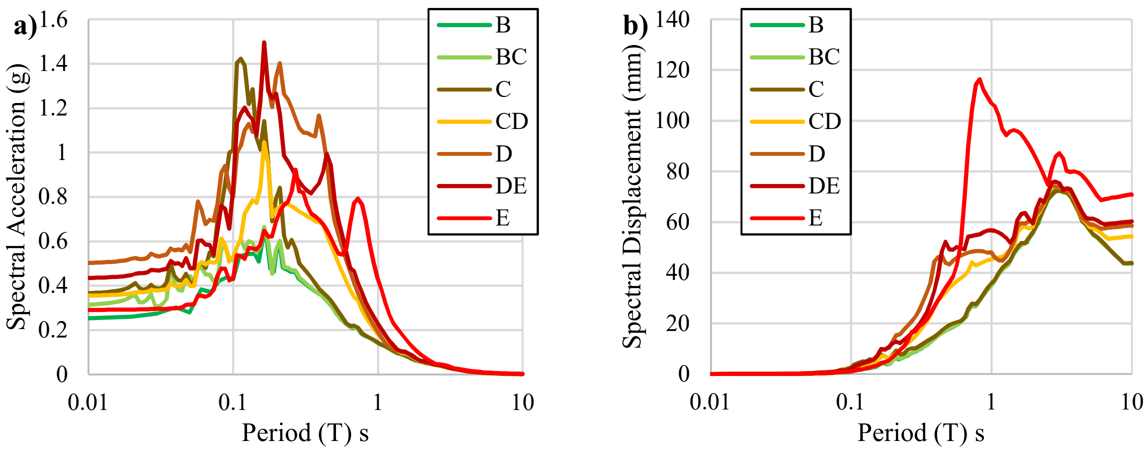

Figure 5 2500-year return period spectra (a) acceleration response and (b) displacement response acceleration response

4. Building capacity

Capacity curves for the building stock and for the purpose of seismic vulnerability studies are commonly generated from generic building parameters. For example, previous seismic vulnerability studies for Australia have relied on adopting building parameters from other regions to model the capacity of structures (Daniell *et al.*, 2015; Koschatzky & de Oliveira, 2016; Koschatzky *et al.*, 2015a, 2015b). Instead, a plastic hinge analysis (PHA) can be used to derive the force-displacement capacity curves of each building. The authors have recently developed or modified some of the PHA expressions for RC walls that better represent the types of structures found in Australia and other low-to-moderate seismic regions (e.g. lightly reinforced and unconfined walls) (Hoult *et al.*, 2017a, 2017b, 2017c). These expressions will provide a more realistic result of the displacement capacity (and capacity curve) of the different RC structural wall buildings in Australia.

The Census of Land Use and Employment (CLUE) dataset (Melbourne City Council, 2015) is a valuable research tool providing comprehensive building information, including construction year, number of storeys ("n", above ground), building material, location (latitude and longitude) and gross floor area. Table 2 gives the total number of low-rise (LR) ($2 \leq n \leq 3$), mid-rise (MR) ($4 \leq n \leq 7$) and high-rise (HR) ($8 \leq n \leq 12$) "concrete" buildings that will be used from the CLUE dataset for the seismic assessment in this research investigation. The total number of buildings in Table 2 is also mapped with their corresponding location in Figure 2(b).

Table 2 Number of buildings used from the CLUE dataset (Melbourne City Council, 2015)

	LR	MR	HR
Number of buildings	821	363	219

Different Building Types, varying by the use of rectangular and/or C-shaped RC walls for the lateral load resisting elements, are to be used in representing the idealised buildings for the city of Melbourne. Other researchers have followed similar methods in idealising the RC building stock for seismic performance studies (Hancock & Bommer, 2007; Lestuzzi & Bachmann, 2007; Mwafy & Elnashai, 2001; Surana *et al.*, 2015). Four building configurations will be used in this study: Type 1, Type 2, Type 3 and Type 4, which are illustrated in Figure 6. Only particular building types can be used to represent the low-rise, mid-rise and high-rise structures, which are dependent on the number of storeys; this is because the buildings will be initially designed for earthquake loading (using AS 1170.4) and/or wind loading (using AS 1170.2), depending on the year of construction. For example, a high-rise building may not have the (moment) capacity for the earthquake or wind demand if it only has C-shaped centralised walls (building Type 3). Therefore, HR buildings are limited to Type 4. Moreover, the single C-shaped wall building (Type 2) is limited to LR buildings designed pre-1995, before earthquake loading became a design requirement. This is because the wind loading requirement for LR buildings is typically small, and it would be unlikely that these types of buildings have the capacity when considering earthquake loading (due to the extra base shear caused from the expected torsional response). It should be noted that it is assumed for all buildings that centre of stiffness provided by the lateral load resisting walls for each principle direction is close to the centre of mass; therefore, the effects of torsional displacement due to in-plane asymmetry have been neglected in this study. It should also be emphasised that the HR buildings investigated here have a 12-storey limit as buildings taller than this are likely to have higher mode effects not captured by the capacity spectrum method. Moreover, LR buildings that are

1-storey high have not been included in the analyses due to the low height of the building (and corresponding cantilever walls); only walls with an aspect ratio greater than 2 have been used in this research. Furthermore, for this study, the C-shaped walls are assumed to be uncoupled. This assumption is only valid for moderate "high-rise" structures (less than 13-storeys), since a coupled and stiffer centralised core (boxed section) would be typical for very tall structures.

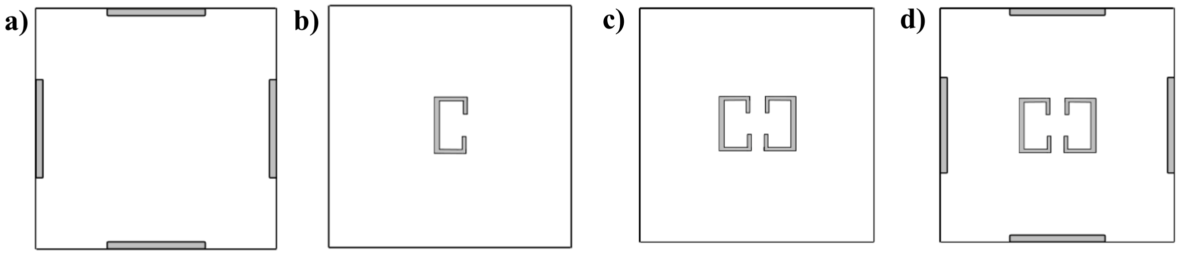

Figure 6 The different idealised building configurations used for RC buildings in Australia (a) Type 1 (b) Type 2 (c) Type 3 and (d) Type 4

The range of values used for some of the building parameters in the MATLAB assessment program are summarised in the Table 5, Table 6 and Table 7 given in Appendix A. Many of these parameters, such as material properties, are selected at random from a generated number based on a normal distribution (if a mean and standard deviation can be provided) or are randomly chosen between an appropriate minimum and maximum range. In contrast, some other parameters, such as the axial load ratio (*ALR*), are randomly chosen between a minimum and maximum value; in the case of the *ALR*, a minimum of 0.01 (1%) and a maximum of 0.1 (10%) is used, based on common values used in previous research (Henry, 2013) and investigations by Albidah *et al.*(2013) for low-to-moderate seismic regions. It should be noted that other seismic assessment methodologies, such as HAZUS (FEMA, 1999) and EQRM (Robinson *et al.*, 2005), also incorporate variability of the building stock through lognormally distributed capacity functions that are calculated based on a chosen, random number.

5. Seismic assessment

An assessment program was written in MATLAB to find the expected damage distribution (Collapse Prevention) for 500-year and 2500-year return period earthquake events for the Melbourne CBD using the capacity spectrum method. The resulting number of buildings that reach or exceed the Collapse Prevention performance level, for the 1403 RC structural wall buildings that have been analysed in this study, are 34 (2.4%) and 540 (38.5%) for the 500-year and 2500-year return period respectively. It should be noted that "Collapse Prevention" corresponds to strain limits in the extreme tension (steel) and compression (unconfined concrete) fibre regions of 0.05 and -0.003 for this research investigation. Table 3 gives the values of the LR, MR and HR buildings for these two return period events that make up the total number of buildings that have reached or exceeded the collapse prevention performance level. The locations (latitude and longitude) of these buildings were also output to ArcMap (ESRI, 2013), a geospatial processing program. The buildings were mapped atop of the different site classes from McPherson (2017) and are given in Figure 7(a) and Figure 7(b) for 500-year and 2500-year return periods respectively. It is interesting to note that 32 of the 34

RC buildings predicted to reach or exceed the Collapse Prevention performance level are (primarily LR structures) located on soil class *D*, *DE* or *E* (e.g. "soft" soils) [Figure 7(a)].

Table 3 Number of buildings reaching or exceeding Collapse Prevention

	Return Period (years)	
	500	2500
LR	31	369
MR	3	125
HR	0	46
Total	34	540

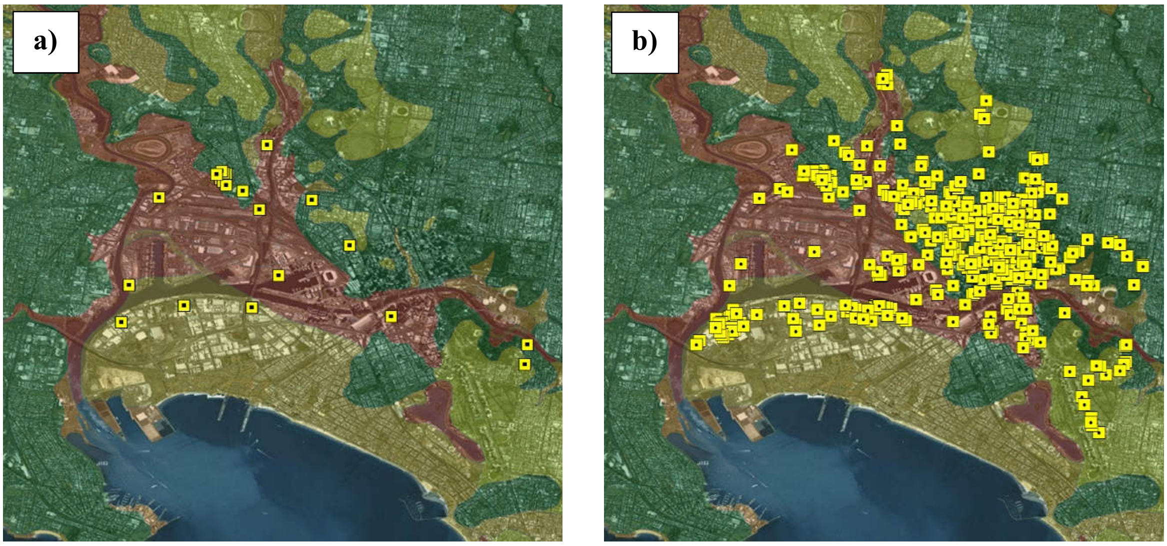

Figure 7 Damage distribution (Collapse Prevention) for (a) 500-year and (b) 2500-year return period earthquakes

6. Fragility curves

Using a similar methodology to that given in the previous sections, fragility functions have been derived for RC structural wall buildings by assessing the Melbourne concrete building stock. A wide range of different ground motions, and subsequently different intensities, were used to derive these fragility curves; the PEER (2016) Ground Motion Database was used to obtain appropriate acceleration time-histories for the region following the same methodology and criteria found in Hoult *et al*. (2016). Furthermore, GENQKE (Lam, 1999; Lam *et al*. 2000) was used in creating artificial ground motions (acceleration time-histories) using parameters for the southeast Australian region. These acceleration time-histories were also used in SHAKE-2000 to obtain an estimate of the site response. For the sake of brevity, only the preliminary fragility curves (or functions) have been presented in this section, while it is expected that the methodology and ground motions used and subsequent results will be published soon. The preliminary fragility curves for the RC structural wall building stock of Melbourne for LR, MR and HR ($n \leq 12$) buildings are given in Figure 8, Figure 9 and Figure

10 respectively. It should be noted that the *PGV* was converted to MMI using the equations from Gaull *et al.* (1990).

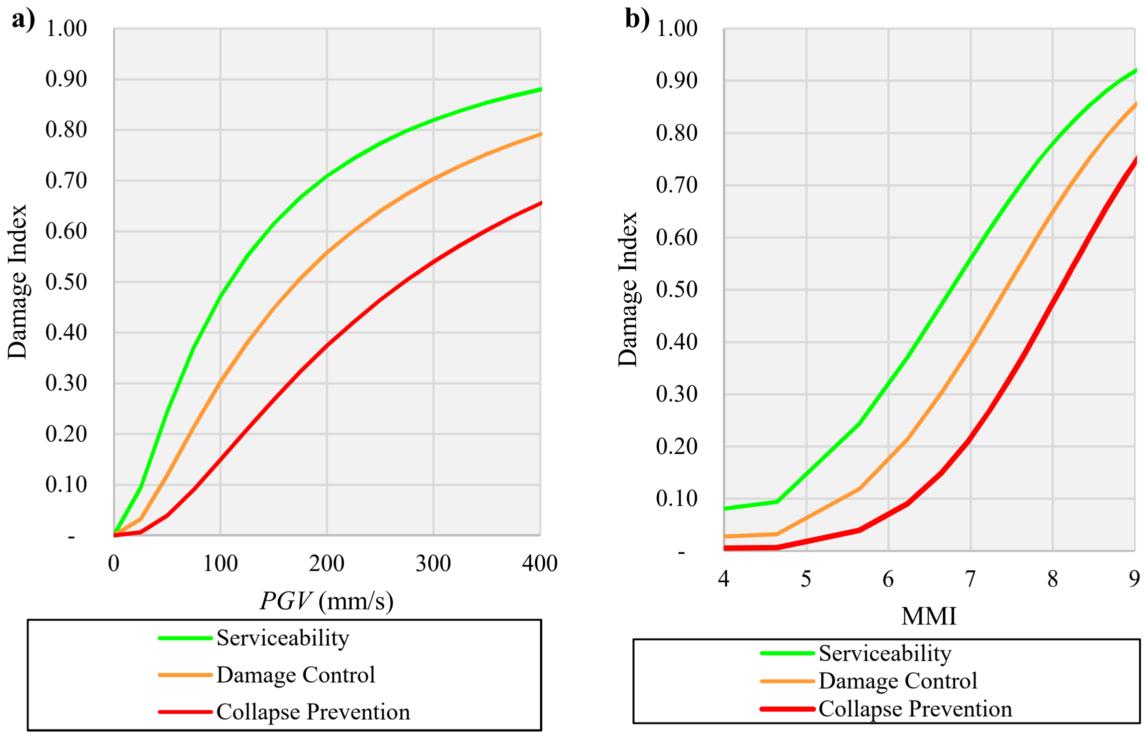

Figure 8 Fragility curve results for LR RC structural wall buildings for an intensity measure of (a) *PGV* and (b) MMI

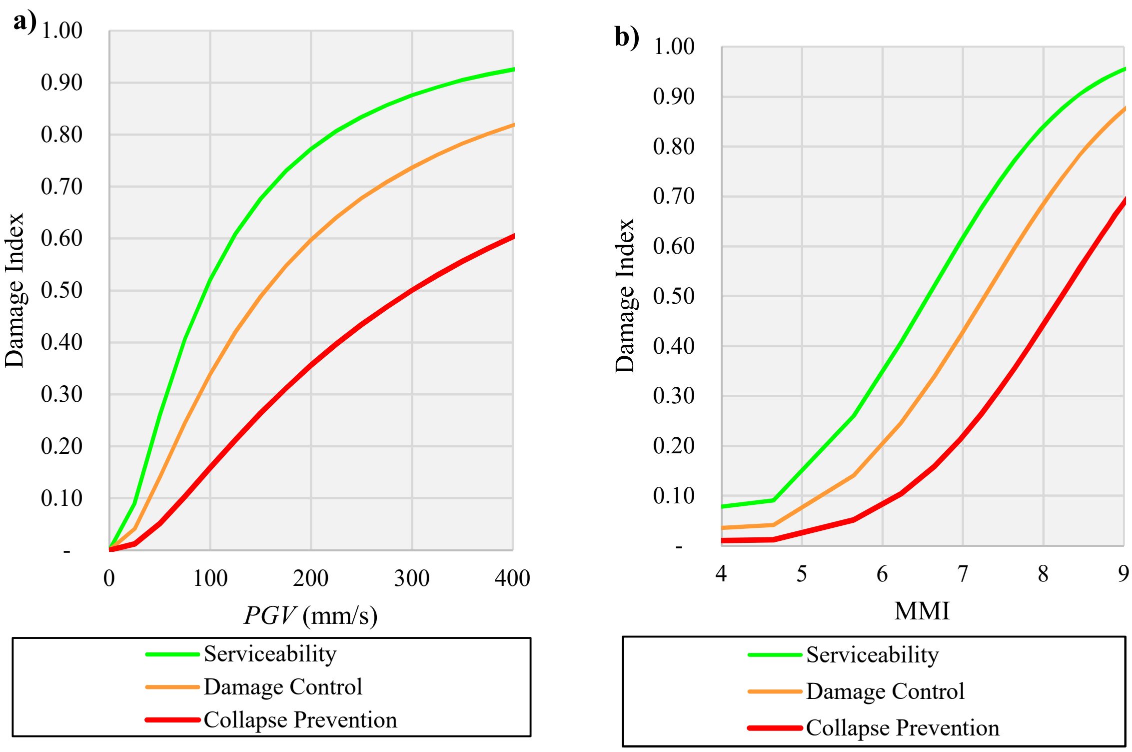

Figure 9 Fragility curve results for MR RC structural wall buildings for an intensity measure of (a) *PGV* and (b) MMI

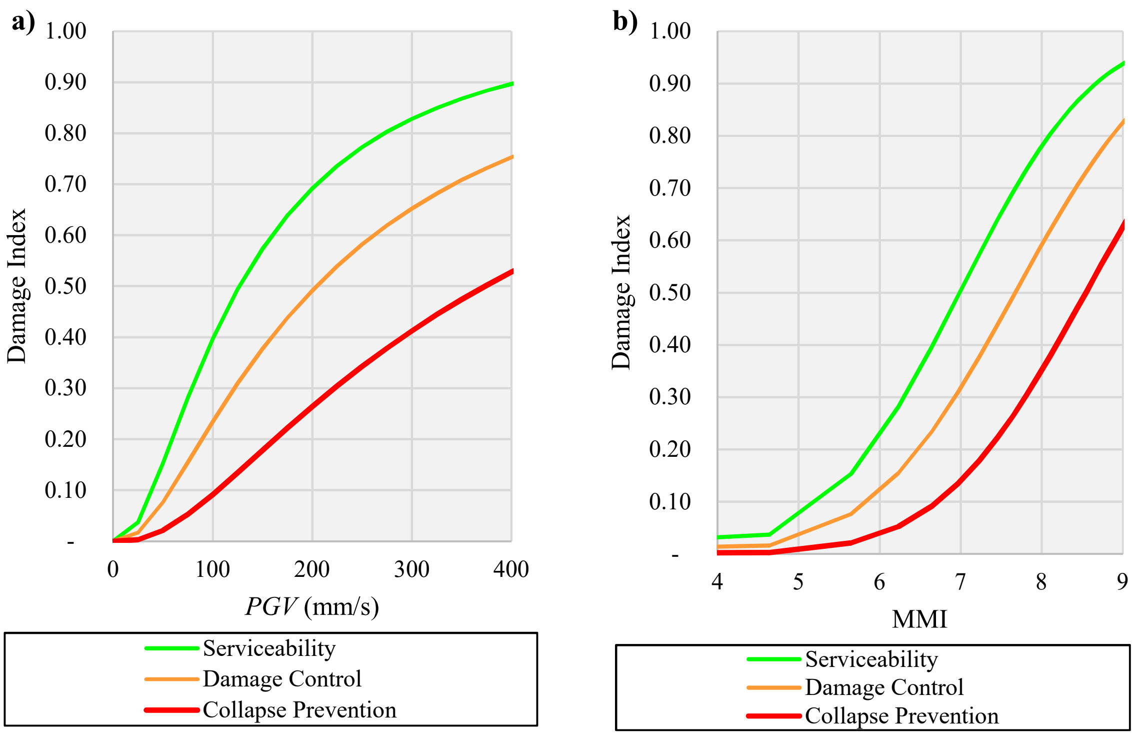

Figure 10 Fragility curve results for HR RC structural wall buildings for an intensity measure of (a) *PGV* and (b) MMI

Although the results from this study (Figures 8 – 10) are specific to the RC shear wall building stock of Melbourne, the observed damage distributions from the 1989 Newcastle earthquake can be used for some comparisons to the results here. The main event was estimated to be of local magnitude (M_L) 5.6 (McCue *et al*., 1990). No strong ground motion recording of the main event exists as there were no instruments installed close to the epicentre of the Newcastle earthquake at the time of rupture (Chandler *et al*., 1991; Melchers, 1990). However, Melchers (1990) used attenuation functions to predict that the *PGV* of the main event was within 30 mm/s to 100 mm/s. Furthermore, the synthetic ground motions predicted by Sinadinovski *et al*. (2000) to replicate the Newcastle main event estimated *PGV* values within the range of 40 mm/s to 50 mm/s. If a value of *PGV* of 50 mm/s is assumed to correspond to the Newcastle main event, the result using Figure 8(a) predict that approximately 24%, 12% and 4% of LR RC shear walls buildings would reach or exceed the performance levels of Serviceability, Damage Control and Collapse Prevention (respectively) in such an event. In the research conducted by Chandler *et al*. (1991), it was observed that approximately 19%, 10% and 3% of (commercial) ‘RC Frame’ buildings reached “damage levels” of D4, D3 and D2, the large majority of which were LR structures. Considering that the definitions of the different damage levels from Chandler *et al*. (1991) (given in Table 4) correlate closely to the definitions of the performance levels used in this research, it is interesting to note the close correlations of damage index from the estimates of this research to the observations from the Newcastle earthquake. However, the isoseimsal map from the main event earthquake (Melchers, 1991) indicate intensities of (at least) MMVI in Newcastle. Using this intensity value would produce larger estimates of damage using the results in Figure 8(b) in comparison to the *PGV*. It is likely that this moderate magnitude earthquake event was of short duration (of high intensity) with most of the frequency content being shifted towards the shorter periods (e.g. high *PGA*, low *PGV*). This is a common characteristic of Australian earthquakes (Sinadinovski *et al*., 2000) and could explain the large intensities felt from the Newcastle main event. The duration is certainly consistent with the observations, where the event was described to last no more than 3 seconds (Chandler *et al*., 1991; McCue *et al*., 1990) and there were even estimates of the ground motions lasting 1.5 to 2 seconds (Melchers, 1991). The M_L 5.4 Moe earthquake event in 2012 is another example of a high *PGA*, low duration event, with most of the frequency content being shifted towards the short period range (captured on five seismometers within 100 kilometres) (Hoult, 2017).

As stated previously in this section, the results from the analyses reported in this paper is specific to the RC shear wall building stock of Melbourne. Therefore, the comparisons to the damage index observed from Newcastle is admittingly a crude estimating, primarily due to (i) differences in building stock, (ii) soil profiles used for demand in the analyses specific to the Melbourne region and (iii) M-R combinations used in the analyses specific to the Melbourne CBD.

Table 4 Definition of damage levels (Chandler *et al*., 1991)

	Damage State	Definition
D0	Undamaged	No visible damage
D1	Slight Damage	Infill panels damages
D2	Moderate Damage	Cracks < 10 mm in structure
D3	Heavy Damage	Heavy damage to structural members, loss of concrete
D4	Partial Destruction	Complete collapse of individual structural member or major deflection to frame
D5	Collapse	Failure of structural members to allow fall of roof or slab

7. Conclusion

The number of RC structural wall buildings reaching or exceeding the Collapse Prevention performance level for a 500-year return period can be considered reasonable. This is "reasonable" in the sense that a large percentage of the RC structural wall building stock analysed here was estimated to be constructed before 1995 (725 of 1403 buildings, approximately 52%) and thus are expected not to have been designed for earthquake actions. For the buildings that have been constructed post-1995, it is likely that the typical 500-year return period earthquake design would have been considered, corresponding to buildings of 'normal importance' (ABCB, 2016). While the great majority of RC structural wall buildings in Australia are estimated to withstand the predicted 500-year return period earthquake event, almost 40% of the building stock analysed for this assessment study reached or exceeded the Collapse Prevention performance level for the 2500-year return period event. Comparing the expected damage distributions from the two return period earthquake events has emphasised the consequences being related non-linearly to the magnitude of the event in areas of dense populations; 'The loss of a hundred lives in one event on average every 10 years has a much greater impact on the community than the loss ten lives on average every year from separate individual events' (Walker & Musulin, 2013). For example, a larger magnitude event than the Newcastle earthquake, but located closer to the Sydney CBD, could cause much greater losses than the 1989 event (deaths, injuries and economically), some estimating it could cause economic losses in the order of AUD$25 billion (Walker, 2008). Furthermore, Pampanin *et al*. (2011) discuss the 'on-off' nature of 'non-ductile' pre-1970s RC structures in Christchurch, which resemble the type of detailing that would be found in RC structures constructed in Australia at presented. It is worth noting that the only two RC structural wall buildings that collapsed in a catastrophic fashion after the Christchurch earthquake event in 2011 were constructed pre-1980s and were 'non-ductile' (Goldsworthy & Gibson, 2012), before the implementation of capacity design principles. Goldsworthy (2012) discusses this further to say that these types of structures would behave reasonable 'until their ductility is exceeded and then [a] dramatic and sometimes totally catastrophic failure' would occur. However, it is acknowledged that research by Kam and Pampanin (2011) estimated that the number of RC buildings built prior to 1990 that reached Collapse Prevention from the Christchurch earthquake was 1.3%, 7.1% and 15.0% for LR, MR and HR buildings, which does not correlate with the fragility functions in Figures 8 – 10; for example, LR and MR RC shear wall buildings were found to be more vulnerable in this research in comparison to HR RC shear wall buildings. Nevertheless, it is possible that many of the buildings that were studied by Kam and Pampanin (2011) had 'capacity design' principles employed, which is not considered in current Australian engineering practice.

The research findings here emphasise the same performance for RC structural wall buildings in Australia, where the following combination of things lead to the great majority of the RC structural wall building stock having the reserve to resist the 500-year return period earthquake event, but not the 2500-year return period event: (i) the potential for a single-crack failure in lightly reinforced walls, (ii) 'normal' ductile reinforcement being primarily (and currently) used (Standards Australia/New Zealand, 2001) and (iii) transverse reinforcement at boundary ends to confine concrete commonly not required by the current AS 3600:2009 (Standards Australia, 2009).

8. Acknowledgments

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Appendix A – Building Parameters for Seismic Assessment

Table 5 Building Types with limiting number of storeys

Building Type	minimum *n*	maximum *n*	Rise
1	2	4	low, mid
2	2	3	low
3	2	7	low, mid
4	4	12	mid, high

Table 6 Dimensions of the C-shaped walls

Wall	t_w (mm)	L_{web} (mm)	L_{flange} (mm)	L_{return} (mm)
LR	200	3600	2000	600
MR	200	6200	2200	600
HR	250	8500	2500	600

Table 7 Wall parameters and values considered for the MATLAB program

		Normal Distribution		Random variables		Set Value	
Parameter	Description	mean	standard deviation	minimum	maximum		Units
f_y	yield strength of reinforcing steel	551	29.2	500	-		MPa
f_u	ultimate strength of reinforcing steel	660.5	37.65	540	-		MPa
E_s	Young's Modulus of reinforcing steel	-	-	-	-	200,000	MPa
ε_{sy}	strain at yield of reinforcing steel	-	-	-	-	f_y/E_s	-
ε_{sh}	strain hardening parameter of reinforcing steel	0.0197	0.0095	-	-		-
ε_{su}	uniform elongation strain of reinforcing steel	0.0946	0.016	0.03	-		-
κ	increased strength factor for concrete (pre-1981)	1.5	0.4	1.2	-		-
κ	increased strength factor for concrete (post-1981)	1.5	0.2	1.0	-		-
f_{cmi}	mean insitu strength of concrete	-	-	-	-	32κ	MPa
E_c	Young's Modulus of concrete	-	-	-	-	$5000\sqrt{f_{cmi}}$	MPa
ALR	axial load ratio	-	-	0.01	0.1[a]/0.05[b]		-
G	dead Load	-	-	4	8		kPa
Q	live load	-	-	1	4		kPa
h_s	inter-storey height	-	-	3.0	3.5		m
ρ_{wv}	longitudinal reinforcement ratio	-	-	0.19%	1.00%		-

[a] = Rectangular walls
[b] = C-shaped Walls