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IMPROVEMENTS AND DIFFICULTIES ASSOCIATED WITH THE SEISMIC ASSESSMENT OF INFRASTRUCTURE IN AUSTRALIA

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Cover: Earthquake damage in Christchurch, New Zealand, 2011. Photo by Jo Johnson, New Zealand Fire Service

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ABSTRACT

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Australia is in a region of low-to-moderate seismicity, but experiences a higher level of seismic activity than other active intra-plate regions around the world. Because of the low earthquake return period that is typically used in design, coupled with the poor quality of reinforcement detailing that is required by current Standards, it is anticipated that many of the typical reinforced concrete (RC) structures in the Australian building stock have limited ductility. Moreover, it has only been the last couple of decades that structural engineers have been required to consider the forces that are associated with a low return period earthquake event. This paper aims at providing some of the latest research and modelling that can be incorporated in the seismic assessment of a structure in Australia. The seismic demand for a region in Australia is primarily dependent on the models used for the earthquake recurrence, attenuation and the site response. A building's capacity can be found using a displacement-based assessment, where the building can be modelled as an equivalent single-degree-of-freedom (SDOF) structure. Some of the assumptions and parameters involved in the modelling processes for seismic demand and a building's capacity are scrutinized for their validity in places of low-to-moderate seismic regions, such as Australia. Potential vulnerabilities within the building stock of Australia, primarily associated with reinforced concrete wall and core buildings, are discussed.

END USER STATEMENT

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INTRODUCTION

On average Australia experiences two earthquakes that are over magnitude 5 per year (Leonard, 2008) and a magnitude 6 every five years (Wilson *et al.*, 2008a). This corresponds to a higher level of seismic activity than other active, intra-plate regions around the world. Earthquake events such as the M6.8 Meckering in 1968, M5.4 Adelaide in 1954 and three M>6 occurring within a twelve hour period at Tennant Creek in 1988 clearly demonstrate that moderate to large size earthquakes can occur and have the potential to tragically affect Australian communities. The most damaging and costly earthquake in Australia was a moderate magnitude 5.6 earthquake that struck the New South Wales city of Newcastle in 1989. The earthquake caused widespread damage as illustrated in Figure 1. Damages and losses cost up to $4 billion if the event and damage were to recur today, with the earthquake ultimately taking the lives of 13 people (Walker, 2011). Standards Australia delivered an earthquake actions loading provision AS 1170.4-1993 (Standards Australia, 1993) after the Tennant Creek and Newcastle earthquake events. This subsequently required earthquake loading to be part of the general design for structures in all areas of Australia (Wilson *et al.*, 2008a). This means that the older building stock can be more vulnerable to seismic loading in Australia compared to building stock that has been designed in the last couple of decades. It is therefore imperative to understand the seismic performance of the Australian building stock under different levels of earthquake loading. This paper aims to provide some of the latest research and modelling that can be incorporated in the seismic assessment of a structure in Australia.

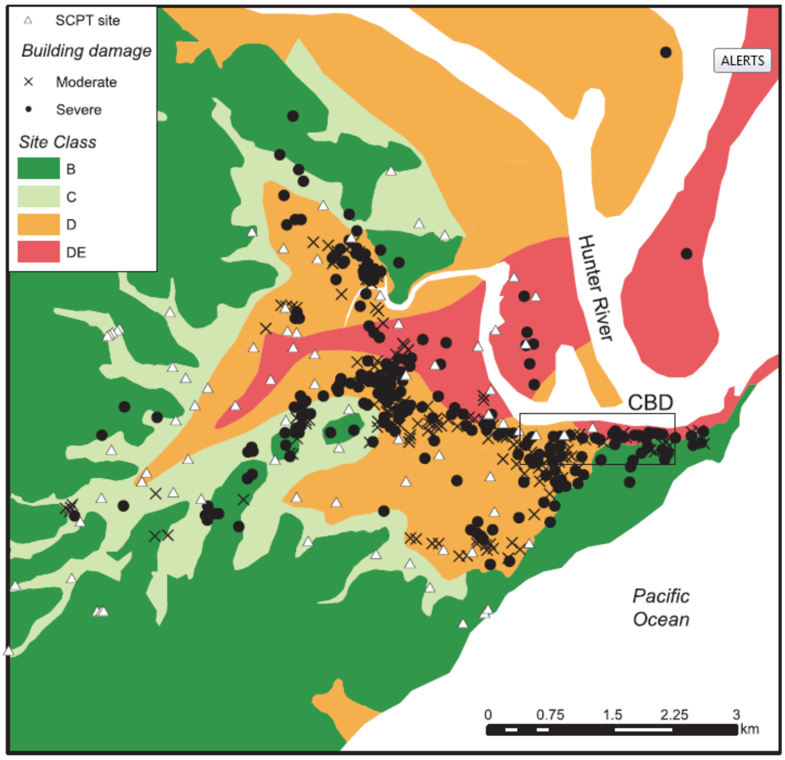

FIGURE 1 SITE CLASSIFICATION MAP FOR THE NEWSCASTLE AREA SHOWING SURVEYED LOCATIONS OF BUILDING DAMAGE FROM THE 1989 NEWCATSLE EARTHQUAKE (MCPHERSON & HALL, 2013)

BACKGROUND

SEISMIC HAZARD IN AUSTRALIA

The seismic demand is primarily dependent on the models used for the earthquake recurrence, attenuation and site response. There have been some improvements over the past decade or so in attempting to quantify the seismic hazard of the different regions in Australia.

EARTHQUAKE RECURRENCE MODELLING

The recurrence model which was developed by Geoscience Australia (GA) was subsequently used in creating the recently released 2012 Australian Earthquake Hazard Map (Burbidge, 2012; Leonard *et al.*, 2013a). The new map offers updated seismic hazard values throughout the country and is thought to be an improvement of the current map developed in 1993 by McCue *et al.* (1993), that is still used in the current AS 1170.4 (Standards Australia, 2007). The new hazard values, equivalent to the peak ground acceleration (PGA) on rock for a 500-year return period, have decreased for most capital cities in Australia. Interestingly, the 2500-year return period PGA is generally higher for the capital cities, with the probability factor (k_p) differing significantly for each capital city in Australia from what is currently given in AS 1170.4 (Standards Australia, 2007). However, it should be noted that although the 2500-year return period hazard value (or PGA) from GA is slightly higher than what is currently stipulated by the Australian Standards, the resulting spectrum for the period range is much lower. Figure 2 gives the resulting 500 and 2500 year return period spectra for Melbourne from AS 1170.4 (Standards Australia, 2007), GA (Leonard *et al.*, 2013a) and AUS5 (Hoult, 2014) out to a period of 1.0s.

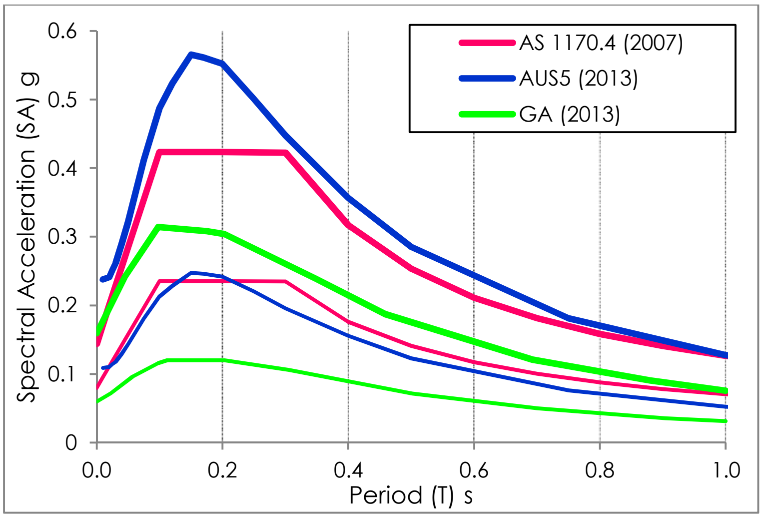

FIGURE 2 COMPARISON OF THE 500 YEAR (THIN LINES) AND 2500 YEAR (THICK LINES) RETURN PERIOD SPECTRA

The AUS5 recurrence model developed by Brown and Gibson (2004) is another seismotectonic model for Australia that is based on numerous layers of geological, geophysical and seismological information, and assumes a relationship between the current seismicity, with geology and the past and present tectonics. Many other earthquake recurrence models assume widely uniform seismicity, which tend to give a much lower hazard for "active" regions, while models that are based on known seismicity tend to give higher values. The AUS5 model, with smoothed seismicity, lie in between the two extremes (Gibson & Dimas, 2009). Hoult (2014) undertook a study using the AUS5 model to determine the seismic hazard for most capital cities in Australia. One of the outcomes of the study resulted in higher k_p values for all cities in Australia for the 2500-year return period event, similar to the findings from GA (Leonard *et al.*, 2013a). These values are compared for each city in Table 1. This has also been found by other researchers (Bull, 2008; Nordenson & Bell, 2000; Tsang, 2006). Overall, there is recognition of the larger ratio of seismic ground motions experienced in a very rare earthquake return period event (e.g. 2500-years) to that experienced in a 500-year return period event for places of low-to-moderate seismicity in comparison to places of high seismicity. The current Building Code of Australia (ABCB, 2015) specifies a 500-year return period for buildings of importance level 2, which is the standard design return period for earthquake actions in Australia for ordinary buildings. The implications of this could result in higher probabilities of structural collapse in low-to-moderate seismic regions, such as Australia, compared to high seismic regions when subjected to the very rare earthquake event.

	Probability Factor (k_p)		
Location	**AS 1170.4 (Standards Australia, 2007)**	**GA (Leonard *et al.*, 2013a)**	**AUS5 (Hoult, 2014)**
Adelaide	1.80	2.69	2.18
Brisbane	1.80	3.05	3.31
Melbourne	1.80	2.62	2.36
Perth	1.80	2.67	2.09
Sydney	1.80	2.83	2.08
Canberra	1.80	2.77	2.14
Hobart	1.80	3.01	3.09

TABLE 1 PROBABILITY FACTOR (K_P) COMPARISONS FOR A 2500-YEAR RETURN PERIOD

The National Earthquake Hazards Reduction Program (NEHRP) 1997 provisions recognised the margin against collapse integrated in design procedures represented by the hazard values for 2500-year return period levels multiplied by two-thirds (Nordenson & Bell, 2000). This resulted in a similar hazard value to the 500-year return period in areas of high seismicity, but an increase 'as much as 100 to 200 % greater in areas of low to moderate seismicity' (Nordenson & Bell, 2000). Increasing the design return period from 500-years to a 2500-year return period has already been a note of consideration for Standards Australia and the ABCB with the next revision of the earthquake loading standard (Wilson *et al.*, 2008b). Furthermore, when the 2012 Australian Earthquake Hazard Map was first introduced at the Australian Earthquake Engineering Society meeting, there was a general consensus to increase the reference hazard return period (or probability of exceedance) given in the Building Code of Australia (BCA) for buildings of normal importance to 2500 years (Leonard *et al.*, 2013b).

A recent study by Leonard *et al.* (2014) compared the different earthquake recurrence models for Australia; the GA and AUS5 models give quite different seismic hazard values of 0.059 and 0.109 respectively for a 500-year return period earthquake event in Adelaide. The variation of seismic hazard using the different recurrence models was shown to be primarily a function of the different recurrence estimations and how faults are included. Another important parameter that can affect seismic hazard studies is the choice of ground motion prediction equations (GMPEs).

SEISMIC ATTENUTATION IN AUSTRALIA

The attenuation of the seismic ground motions through the crust can be estimated using GMPEs, where typical input parameters include the magnitude of the earthquake and the distance of the site from the epicentre. The number of high-quality ground motion recorders in Australia is limited and the catalogue of recorded earthquake events in Australia is sparse, which make it particularly difficult to develop accurate attenuation models for Australian conditions (Burbidge, 2012). There have been some attempts in deriving GMPEs for the different regions of Australia, but the lack of strong motion data for Australia makes it difficult to validate these models and forces seismologists and earthquake engineers to also adopt GMPEs from other regions around the world with similar geology (Hoult *et al.*, 2013). Adopting GMPEs is an inviting approach, as some functions that have been derived using an abundance of data (high seismic regions) with similar geology may be applicable to some regions of Australia. Brown and Gibson (2004) believe that the GMPEs developed in western North America, such as the Next Generation Attenuation (NGA-West 2) functions, are more applicable to eastern Australia compared to the models developed for eastern North America (stable, intra-plate region). Depending on the function there can be quite a large variability in estimated attenuation and resulting acceleration (and displacement) response, as shown in Figure 3 for various GMPEs used by GA (Burbidge, 2012) for the 2012 Australian Earthquake Hazard Map.

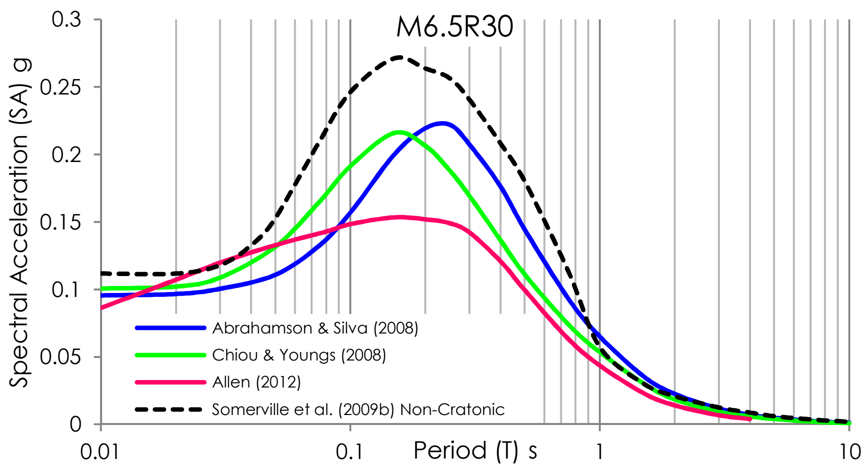

FIGURE 3 THE RESULTING ACCELERATION RESPONSE SPECTRA FOR A MAGNITUDE 6 AT A SITE 30KM FROM THE EPICENTRE USING VARIOUS GMPEs

The 2012 Moe earthquake event (M5.4) and main aftershock (M4.4) in Victoria provided many recordings of the events at close and varying distances from the epicentre. Using the Moe earthquake data, Hoult *et al.* (2014a) used the predictions from potentially applicable GMPEs for eastern Australia to try and validate their use for the region. 'This can be particularly important for hazard studies that utilise Probabilistic Seismic Hazard Analyses (PSHAs), in which there is a high dependency on the type of ground motion models used' (Hoult *et al.*, 2014a). The results tend to show some agreement with the earlier observations made by Allen and Atkinson (2007), that the attenuation of seismic motions of south eastern Australia attenuates in a similar way to regions of eastern North America within short distances from the epicentre. However, some of the results also infer that the functions developed in western North America are more applicable for eastern Australia. The study was inconclusive and further research is necessary to either develop or assess applicable GMPEs for different regions in Australia, since a larger dataset is needed for a statistically meaningful study. This will ultimately improve the accuracy of earthquake hazard studies in Australia.

SITE RESPONSE

Site response also plays an important role in estimating the earthquake demand for a building. It has widely been accepted in the engineering community that the seismic motions at the surface of a soil deposit can be significantly different to the seismic motions of the underlying rock. The general view, which is consistent with the current AS 1170.4 (Standards Australia, 2007), is that the harder the rock the less amplification of the ground motion at the surface compared with bedrock motion. Conversely, the softer the soil deposit the greater the amplification of the ground motion at the ground surface. However, recent studies have shown results that contradict this generally accepted representation of site response. A parametric study of different soil profiles by Dhakal *et al.* (2013) revealed that two variables significantly affected the seismic excitations at the surface: the shear-wave velocity of the soil (stiffness of the soil) and the intensity of the bedrock motions. The latter of these findings is of most interest, with Figure 4 indicating that lower intensity bedrock motions cause greater amplifications of the response at the surface.

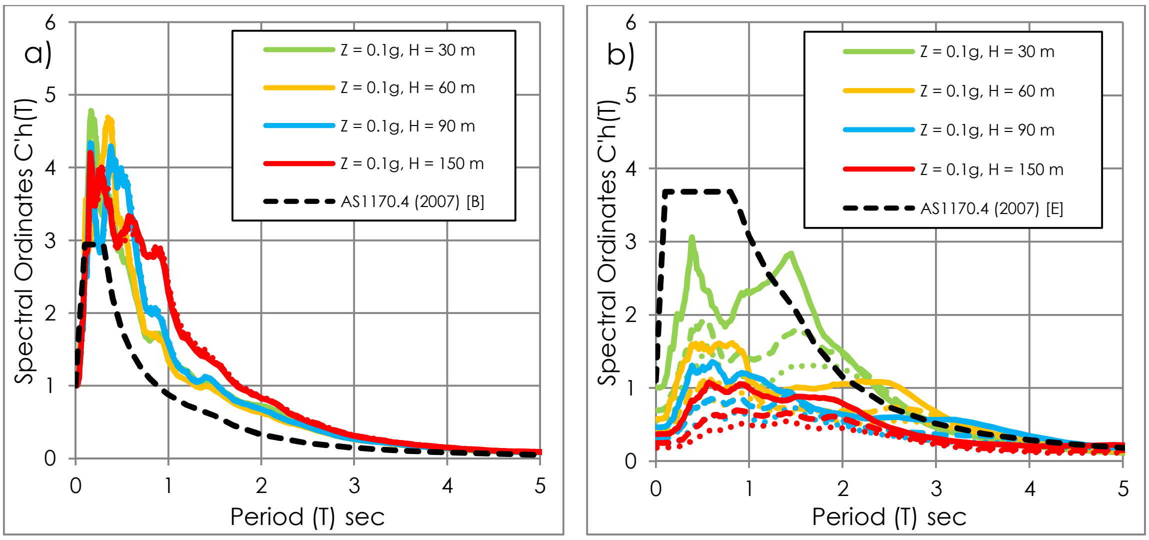

FIGURE 4 THE EFEFCT OF BEDROCK MOTION (Z) INTENISTY OF 0.1g (SOLID LINE), 0.2g (DASHED LINE) AND 0.3g (SQUARE DOT LINE) ON (a) ROCK AND (b) SOFT SOIL

Research conducted at the University of Melbourne (UoM) by the authors, with a primary focus on places of low-to-moderate seismicity such as Australia, found the same intensity dependent amplification. This could be crucial for estimating the earthquake demand in Australia, as the majority of the seismic events in the region are considered to be of low intensity and thus higher amplification of response at the surface would be possible. For example, it is possible to draw correlations between the areas of maximum damage from the Newcastle earthquake in 1989 and the geographical extent of Quaternary sediments (Jones *et al.*, 1996), shown in Figure 1. However, other researchers have argued that the observed damage distribution is mainly controlled by the age and construction type of the building, rather than the correlation between site class and damage (McPherson & Hall, 2013).

The output of the seismic demand typically result in an acceleration or displacement response spectrum format, which can then be used by engineers to compare with the capacity of a building in resisting the predicted ground motions.

ASSESSMENT OF THE CAPACITY

Accurately assessing a structure using a force-based approach can be challenging. The standard force-based assessment is typically based on a simple comparison of the base shear capacity and the base shear demand, where there is no assessment of the actual displacement or ductility capacity and no consideration of risk levels (Priestley *et al.*, 2007). The displacement-based approach can be used to determine the displacement capacity of the structure and to compare it directly with the displacement demand, leading to an estimate of the risk. This approach is far more transparent than force-based methods because damage can be directly related to displacement. The aim is to assess the structure as to whether it has achieved a specified deformation state under a specific design-level earthquake event. However, there are many assumptions made in this approach that may not be applicable to typical building stock in low-to-moderate seismic regions like Australia.

PERFORMANCE OBJECTIVES AND STRAIN LIMITS

Moment-curvature analysis of sections is a simple tool that can be used in determining the force-displacement relationship of RC structures. Curvatures ($\emptyset$), which are rotation per unit length, can be calculated from the analysis and can be used to find the overall deformation of the structure; this is shown in Equation 1 for calculating the displacement (Δ_i) at level *i* as a function of the yield (Δ_y) and plastic (Δ_p) displacement of an RC cantilever wall section, as taken from Priestley *et al.* (2007).

$$\Delta_i = \Delta_y + \Delta_p = \frac{\emptyset_y H_i^2}{2}\left(1 - \frac{H_i}{H_n}\right) + (\emptyset_{ls} - \emptyset_y) L_p H_i \qquad (1)$$

where H_i is the height of the wall at level *i*, H_n is the height at roof level, $\emptyset_{ls}$ is the limit state curvature and L_p is the plastic hinge length.

Using Equation 1, it is possible to determine different limit state displacements for a range of performance objectives; namely, Serviceability, Damage Control and Collapse Prevention. This is done by either limiting the interstory drifts which cause damage to non-structural components or by limiting the curvature at the base of the wall which ensures that compressive and tensile strains in the reinforced concrete section are limited to values approximate to the given limit state. Ultimately, this can be used to produce a capacity curve that can be used in an acceleration-displacement response spectrum (ADRS) format (by dividing the base shear by the mass of the structure). The ADRS format can be used to predict if a building will reach or exceed any of the performance objectives for a given return period earthquake event, illustrated in Figure 5.

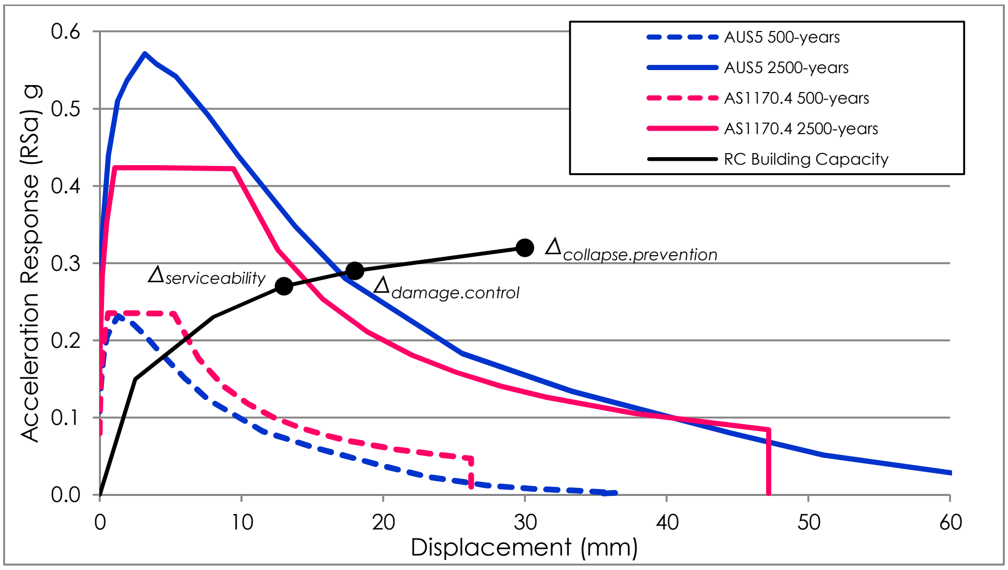

FIGURE 5 CAPACITY CURVE IN ADRS FORMAT WITH PREDICTED 500 AND 2500-YEAR RETURN PERIOD EARTHQUAKE EVENTS FROM AS 1170.4 (2007) AND AUS5 FOR MELBOURNE ON SITE CLASS B

Limiting material strain values need to be determined which correspond to the onset of the different performance objectives. The critical strain values given in Priestley *et al.* (2007) for the different performance objectives assume that the RC sections are well-confined (transverse ties) and have a higher amount of longitudinal reinforcement in comparison to typical RC sections in Australia. Not only would this allow a higher concrete axial strain, but the longitudinal bars are also well restraint by the transverse ties and less likely to buckle once the outer concrete has spalled off. Due to these considerations, Hoult *et al.* (2014b) attempted to define limiting strain values for the different performance objectives for unconfined RC sections, given in Table 2.

The material strain limits presented in Table 2 will ultimately determine the displacement capacity of the structure. However, as Equation 1 shows, the plastic displacement of the structure is also highly dependent on the plastic hinge length (L_p) value.

Structure Performance Limit State (Unconfined Concrete)	Concrete Strain (ε_c)	Steel Strain (ε_s)	Drift Limits (%)
Serviceability: 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.	0.0010	0.005	0.5
Damage Control: 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.	0.0015	0.010	1.5
Collapse Prevention: Ultimate limit state of concrete at spalling 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.	0.0030	0.015	-

TABLE 2 LIMITING STRAIN AND DRIFT VALUES FOR UNCONFINED CONCRETE SECTIONS

PLASTIC HINGE LENGTH

Some RC walls with a light amount of longitudinal reinforcement detailing have been observed to perform poorly in past earthquake events (CERC, 2012; Henry, 2013; Wood *et al.*, 1991; Wood, 1989). There were several cases of a single crack forming at the base in the plastic hinge region after the Christchurch earthquake in 2011, with some of the longitudinal reinforcing bars prematurely fracturing that crossed this crack. This was due to the large concentration of inelastic behaviour over such a small height of the wall (CERC, 2012). Research which focused on one of these walls in the Gallery Apartments building in Christchurch concluded that there was an insufficient amount of longitudinal reinforcement (ρ_{wv}) to initiate secondary cracking (Henry, 2013; Henry *et al.*, 2014; Sritharan *et al.*, 2014). This could be a major issue for low-to-moderate seismic regions, such as Australia, where it is expected that a great percentage of the RC building stock typically incorporates a low amount of longitudinal reinforcement (Hoult *et al.*, 2014b; Wibowo *et al.*, 2013). The minimum longitudinal reinforcement ratio in the current concrete materials standards AS 3600 (Standards Australia, 2009) in Australia is 0.15%, which is similar to the 0.16% detailed in the failed wall of the Gallery Apartments building. In light of the recent observations, research has been conducted at the University of Melbourne by the authors to investigate if the current minimum reinforcement ratio employed in provisions for low-to-moderate seismic regions was adequate in ensuring that some ductility would be achieved.

A simple mathematical model was developed to estimate the amount of longitudinal reinforcement (ρ_{wv}) necessary to allow secondary cracking. This was derived by calculating the amount of reinforcement required to produce a tensile force in the steel large enough to exceed the tensile strength of the concrete that surrounds the bars. This would allow secondary cracking to occur above the primary crack at the base and thus allow the wall to behave in a ductile manner. A validated finite element modelling (FEM) program complimented the mathematical model, with numerical analysis being conducted on a number of different walls with a range of ρ_{wv}. Figure 6 shows the cracking distribution results from the FEM program VecTor2 (Wong & Vecchio, 2002) for the same wall with ρ_{wv} of 0.7% and 0.75%. This conforms to the mathematical model, which estimated that the wall needed a ρ_{wv} of 0.76%

to initiate secondary cracking in the plastic hinge region close to the base where the inelastic rotations occur.

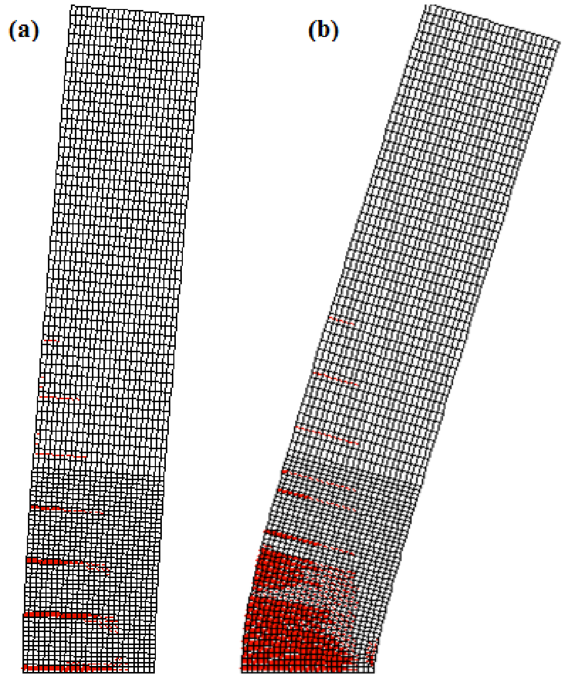

FIGURE 6 CRACKING DISTRIBUTION RESULTS FROM VECTOR2 AT ULTIMATE DISPLACEMENT WITH (a) ρ_{wv}=0.70 % AND (b) ρ_{wv}=0.75 %

The FEM results confirmed that a minimum ρ_{wv} was necessary to initiate secondary cracking in the concrete. This limit has been found to be much higher than the current minimum stipulated in some codified provisions, including AS 3600 (Standards Australia, 2009). Using the mathematical model, a minimum longitudinal reinforcement ratio ($\rho_{wv.min}$) is derived in Equation 2.

$$\rho_{wv.min} = \frac{0.54\sqrt{\kappa f'_c}}{f_u} \tag{2}$$

where f'_c is the characteristic concrete strength and f_u is the ultimate tensile strength of the steel. The κ value is used to increase the concrete strength due to the initial variability in the concrete and increase of the strength with age.

The Structural Engineering Society of New Zealand (SESOC, 2011) recommended a $\rho_{wv.min}$ for RC walls following the observations from the Christchurch Earthquake (Equation 3).

$$\rho_{wv.min} = \frac{0.4\sqrt{f'_c}}{f_y} \tag{3}$$

where f_y is the yield strength of the steel.

Many empirical plastic hinge length equations (Bohl & Adebar, 2011; Kowalsky, 2001; Priestley *et al.*, 2007; Thomsen & Wallace, 2004) have been shown to be unsuitable for lightly reinforced walls that do not exhibit secondary cracking.

This has rather large implications for assessing lightly reinforced walls, as the ultimate displacement capacity (Equation 1) is highly dependent on the plastic hinge length (L_p) and can be grossly overestimated by using the existing equations. In the case of the wall from Figure 6, the L_p was effectively zero for ρ_{wv}=0.70%, while the L_p was 2600mm for ρ_{wv}=0.75%. Moreover, the current earthquake loading provision AS 1170.4 (Standards Australia, 2007) assumes that 'limited-ductile' RC walls have a ductility value (μ) of 2, equivalent to the ratio of ultimate to yield displacement. However, this research shows that many of these typical RC walls are likely to have very little ductility, requiring a reassessment .

CONCLUSION

In 1995 it became mandatory for buildings throughout Australia to be designed with some consideration of earthquake loading. Since then there have been many improvements in the general understanding of the earthquake hazard in Australia. However, this paper has highlighted some of the uncertainties that are still involved with predicting the earthquake hazard in a low-to-moderate seismic region such as Australia.

In addition a study has been conducted to determine the likely ductility of RC walls in Australia. The minimum reinforcement required by AS 3600 (Standards Australia, 2009) has been found to be too low and as a result some RC wall and core buildings have been found to be in danger of a non-ductile failure. Building codes in Australia may need to be improved in order to create a more robust building stock and retrofitting of some older buildings may be needed. Further research is being conducted at the University of Melbourne to assess the performance of a range of structures against a “very rare” earthquake event, and it is hoped that this work will provide a basis for some code recommendations.

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REFERENCES

ABCB. (2015). The National Construction Code (NCC) 2015 Buildings Code of Australia - Volume One: Australian Building Code Board (ABCB).

Allen, T. I., & Atkinson, G. M. (2007). Comparison of Earthquake Source Spectra and Attenuation in Eastern North America and Southeastern Australia. *Bulletin of the Seismological Society of America, 97*(4), 1350-1354. doi: 10.1785/0120060206

Bohl, A., & Adebar, P. (2011). Plastic Hinge Lengths in High-Rise Concrete Shear Walls. [Article]. *ACI Structural Journal, 108*(2), 148-157.

Brown, A., & Gibson, G. (2004). A multi-tiered earthquake hazard model for Australia. *Tectonophysics, 390*(1–4), 25-43.

Bull, D. (2008). Earthquake Engineering in Regions of Low-moderate Seismicity.

Burbidge, D. R. e. (2012). The 2012 Australian Earthquake Hazard Map *Record 2012/71*: Geoscience Australia: Canberra.

CERC, C. E. R. C. (2012). Final report: Volume 2: The peromance of Christchurch CBD Buildings. Wellington, NZ.

Dhakal, R. P., Lin, S. L., Loye, A. K., & Evans, S. J. (2013). Seismic design spectra for different soil classes. *Bulletin of the NZ Society of Earthquake Engineering, 46*(2), 79-87.

Gibson, G., & Dimas, V. (2009). *Earthquake Hazard at Newcastle*. Paper presented at the Australian Earthquake Engineering Society 2009 Conference, Newcastle, New South Wales.

Henry, R. S. (2013). Assessment of the Minimum Vertical Reinforcement Limits for RC Walls. *Bulletin of the New Zealand Society for Earthquake Engineering, 46*(2), 88.

Henry, R. S., Ingham, J., & Lu, Y. (2014). *Experimental testing and modelling to address the performance of RC walls during the 2010/2011 Canterbury earthquakes*. Paper presented at the Tenth U.S. National Conference on Earthquake Engineering, Anchorage, Alaska.

Hoult, R. D. (2014). Reevaluation of the Seismic Vulnerability of Typical Reinforced Concrete Shear Wall Buildings in Australia. Retrieved from http://www.abcb.gov.au/en/education-events-resources/students/scholarship/Completed%20Studies.aspx

Hoult, R. D., Amirsardari, A., Sandiford, D., Lumantarna, E., Goldsworthy, H. M., Gibson, G., & Asten, M. (2014a). *The 2012 Moe Earthquake and Earthquake Attenuation in South Eastern Australia*. Paper presented at the Australian Earthquake Engineering Society 2014 Conference, Lorne, Victoria.

Hoult, R. D., Goldsworthy, H. M., & Lumantarna, E. (2014b). *Seismic Performance of Typical C-Shaped Reinforced Concrete Shear Cores in Australia*. Paper presented at the Australian Earthquake Engineering Society 2014 Conference, Lorne, Victoria.

Hoult, R. D., Lumantarna, E., & Goldsworthy, H. M. (2013). *Ground Motion Modelling and Response Spectra for Australian Earthquakes*. Paper presented at the Australian Earthquake Engineering Society 2013 Conference, Hobart, Tasmania.

Jones, T. D., Neville, M. J., Scott, G., & Sinadinovski, C. (1996). *Earthquake Microzonation and the Developement of the Australian Earthquake Loading Standard*. Paper presented at the Paper presented at the Proceedings of the 1996 Australian earthquake Engineering Society Conference, Adelaide.

Kowalsky, M. J. (2001). RC structural walls designed according to UBC and displacement-based methods. [Article]. *Journal of Structural Engineering, 127*(5), 506.

Leonard, M. (2008). One Hundred Years of Earthquake Recording in Australia. *Bulletin of the Seismological Society of America, 98*(3), 1458-1470. doi: 10.1785/0120050193

Leonard, M., Burbidge, D., & Edwards, M. (2013a). Atlas of seismic hazard maps of Australia: seismic hazard maps, hazard crurves and hazard spectra *Record 2013/41*: Geoscience Australia: Canberra.

Leonard, M., Burbidge, D., & Edwards, M. (2013b). Atlas of seismic hazard maps of Australia: seismic hazard maps, hazard crurves and hazard spectra. *Record 2013/41*: Geoscience Australia: Canberra.

Leonard, M., Hoult, R. D., Somerville, P., Sandiford, D., Gibson, G., Goldsworthy, H. M., Lumantarna, E., & Spiliopoulos, S. (2014, Novemeber 21-23, 2014). *Deaggregating the differences between seismic hazard assessments at a single site*. Paper presented at the Australian Earthquake Engineering Society 2014 Conference, Lorne, VIC.

McCue, K., Gibson, G., Michael-Leiba, M., Love, D., Cuthbertson, R., & Horoschun, G. (Cartographer). (1993). Earthquake hazard map of Australia, 1991.

McPherson, A., & Hall, L. (2013). Site Classification for Earthquake Hazard and Risk Assessment in Australia. *Bulletin of the Seismological Society of America, 103*(2A), 1085-1102. doi: 10.1785/0120120142

Nordenson, G. J. P., & Bell, G. R. (2000). Seismic Design Requirements for Regions of Moderate Seismicity. *Earthquake Spectra, 16*(1), 205-225. doi: 10.1193/1.1586091

Priestley, M. J. N., Calvi, G. M., & Kowalsky, M. J. (2007). *Displacement-based seismic design of structures / M. J. N. Priestley, Gian Michele Calvi, Mervyn J. Kowalsky*: Pavia : IUSS Press : Fondazione Eucentre, 2007.

SESOC. (2011). Practice note - Design of conventional structural systems following Canterbury earthquakes. Report prepared for the Canterbury Earthquakes Royal Commission. http://canterbury.royalcommission.govt.nz/documents-by-key/20111221.1908

Sritharan, S., Beyer, K., Henry, R. S., Chai, Y. H., Kowalsky, M., & Bull, D. (2014). Understanding Poor Seismic Performance of Concrete Walls and Design Implications. *Earthquake Spectra, 30*(1), 307-334. doi: 10.1193/021713EQS036M

Standards Australia. (1993). AS 1170.4-1993: Minimum design loads on structures (known as the SAA Loading Code) - Earthquake loads.

Standards Australia. (2007). AS 1170.4-2007: Structural design actions, Part 4: Earthquake actions in Australia.

Standards Australia. (2009). AS 3600-2009: Concrete Structures.

Thomsen, J., & Wallace, J. (2004). Displacement-Based Design of Slender Reinforced Concrete Structural Walls—Experimental Verification. *Journal of Structural Engineering, 130*(4), 618-630. doi: doi:10.1061/(ASCE)0733-9445(2004)130:4(618)

Tsang, H. H. (2006). *Probabilistic Seismic Hazard Assessment: Direct Amplitude-Based Approach.* The University of Hong Kong, Hong Kong, China.

Walker, G. (2011). Comparison of the impacts of Cyclone Tracy and the Newcastle earthquake on the Australian building and insurance industries. [Article]. *Australian Journal of Structural Engineering, 11*(3), 283-293.

Wibowo, A., Wilson, J., Lam, N. T. K., & Gad, E. F. (2013). Seismic performance of lightly reinforced structural walls for design purposes. *Magazine of Concrete Research, 65*, 809-828. Retrieved from http://www.icevirtuallibrary.com/content/article/10.1680/macr.13.00021

Wilson, J., Lam, N., & Pham, L. (2008a). Development of the new Australian earthquake loading standard: EJSE International Ltd 2008.

Wilson, J. L., Lam, N. T. K., & Pham, L. (2008b). Development of the new Australian earthquake loading standard.

Wong, P., & Vecchio, F. (2002). Vector2 and FormWorks User Manual. Department of Civil Engineering, University of Toronto.

Wood, S., Stark, R., & Greer, S. (1991). Collapse of Eight-Story RC Building during 1985 Chile Earthquake. *Journal of Structural Engineering, 117*(2), 600-619.

Wood, S. L. (1989). Minimum tensile reinforcement requirements in walls. *ACI Structural Journal, 86*(5), 582-591.