Framework for Seismic Vulnerability Assessment of Reinforced Concrete Buildings in Australia

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Abstract

The project "Cost-Effective Mitigation Strategy Development for Building Related Earthquake Risk" under the Bushfire and Natural Hazards Cooperative Research Centre (BNHCRC) aims to develop knowledge to facilitate evidence-based informed decision making in relation to the need for seismic retrofitting, revision of codified design requirement, and insurance policy. Seismic vulnerability assessment is an essential component in the project. The objective of this paper is to present the framework for carrying out the vulnerability assessment of limited-ductile reinforced concrete (RC) buildings in Australia, for public review and comments by the Australian earthquake engineering community. This paper is organised into four parts: (1) description of the three broad categories of vulnerable RC buildings; (2) definition and description for each performance level or damage state; (3) seismic hazard and site classification scheme; and (4) hazard-consistent ground motion input for incremental dynamic analysis (IDA).

Introduction

Natural hazard events such as flood and fire have caused significant damage and devastation across Australia in the last decade. Other natural hazards including cyclone, earthquake and tsunami expose human, infrastructure and societal vulnerabilities, and subject the Australian community to considerable impact and loss. However, the expected amount of losses in terms of economic or casualties have not been fully known or adequately investigated.

The Bushfire and Natural Hazards Cooperative Research Centre (BNHCRC) was established in 2013 to conduct research relating to all kinds of natural hazard related risks. The purpose of the BNHCRC is to conduct applied research in collaboration with end-users in order to reduce the risks from bushfire and natural hazards, reduce the social, economic and environmental costs of disasters, and to make contributions to the national disaster resilience agenda.

The project "Cost-Effective Mitigation Strategy Development for Building Related Earthquake Risk" aims to develop knowledge to facilitate evidence-based informed decision making that is relevant to seismic strengthening and retrofitting strategy for existing vulnerable buildings, revision of design requirements for new structures and updating of insurance policy. All building categories in Australia are covered in the project, whilst the focus would be put onto the two most vulnerable categories of buildings, namely unreinforced masonry (URM) structures and non-ductile (or limited-ductile) reinforced concrete (RC) structures, both of which are prevailing across the whole of Australia.

Cost-benefit analysis (CBA) will be used as a standard tool to facilitate informed decision making (Liel and Deierlein, 2013). Apart from developing socio-economic loss models which are relevant to costing, seismic structural analysis is a core part of the project for investigating the vulnerability of different forms of structures. Seismic fragility curves are the essential input information for CBA, whilst state-of-the-art methodology employing incremental dynamic analysis (IDA) and realistic ground motion records are applied for a wide range of return periods (or rates of exceedance) and representative site conditions. Ground motion records have to be selected and retrieved from ground motion databases or simulated artificially based on the stochastic methodology.

The objective of this paper is to present the framework and key parameters related to the vulnerability assessment of limited-ductile RC buildings in Australia, for public review and comment by the Australian earthquake engineering community. The paper covers four main parts: (1) description of the three broad categories of vulnerable RC buildings; (2) definition and description of each performance level or damage state; (3) seismic hazard and site classification scheme; and (4) hazard-consistent ground motion input for IDA.

Categories of Vulnerable RC Buildings

Earthquake-resistant design has been enforced in Australia since 1995. As the building replacement rate is 2% nationally, the majority of RC buildings in Australia have not been specifically designed for earthquake resistance. However, this does not necessarily mean that Australian RC buildings could not sustain the level of earthquake actions that have been stipulated in the Australian Standard AS1170.4-2007, given that the potential seismic performance of a structure would also be dependent on soil conditions, building height and structural form. The design wind load can be greater than the stipulated seismic actions in many cases, which is a common situation amongst regions of low-to-moderate seismicity.

Tall buildings have typically been designed for resisting high wind pressures. Thus, their lateral load resistance is usually not governed by seismic actions. In contrasts low-to-medium rise buildings are considered more vulnerable in an earthquake, and particularly on flexible soil sites where the spectral demand can be amplified by a few times.

Generally speaking, irregular structural configurations would elevate the seismic demand on structural elements. Soft- and/or weak-storeys attracting a larger displacement demand is a common cause of failure in earthquakes all around the world. Discontinuity (or offset) of gravitational load carrying elements would lead to stress concentration at certain locations. Asymmetric distributions of mass and/or stiffness on the floor plan would induce torsional responses, which in turn increase the force and displacement demand on the structural elements that are remote from the centre of stiffness of the building, e.g., around its perimeter.

Hence, the project team (comprising the authors and associated researchers) proposes to undertake detailed investigation of the following three broad categories of RC structures for their seismic vulnerability and subsequent cost-benefits following retrofitting work:

Category 1 buildings are those featuring a soft- and/or weak-storey that would collapse in a column or beam-column joint failure mechanism, especially those with minimal or no RC walls at the soft-storey level. This category of buildings can be sub-divided further into two construction types, namely precast RC columns and in-situ RC columns. An example of a building supported by precast RC columns at the ground floor is given in Figure 1.

Category 2 buildings are those featuring RC walls as the dominant lateral load resisting system, including those with significant discontinuity (or offset) of gravitational load carrying elements. A typical floor plan of an example building of this category is given in Figure 2. Buildings of this category are typically supported by RC walls surrounding lift cores and emergency exit stairwells to form the building core. However, it is not uncommon for residential buildings, which generally have a smaller internal building core (due to the reduced number of lift shafts and stairwells) to require RC walls around the perimeter of the building to provide supplementary support (Figure 3).

Category 3 buildings are those featuring dual RC moment resisting frames (MRFs) and RC walls to act jointly as a lateral load resisting system. A typical floor plan of an example building of this category is given in Figure 4.

Building models shown in Figure 2 and Figure 4 are the slightly modified versions of real buildings constructed in Australia.

Figure 1. Category 1 building with soft- and/or weak-storey in Melbourne.

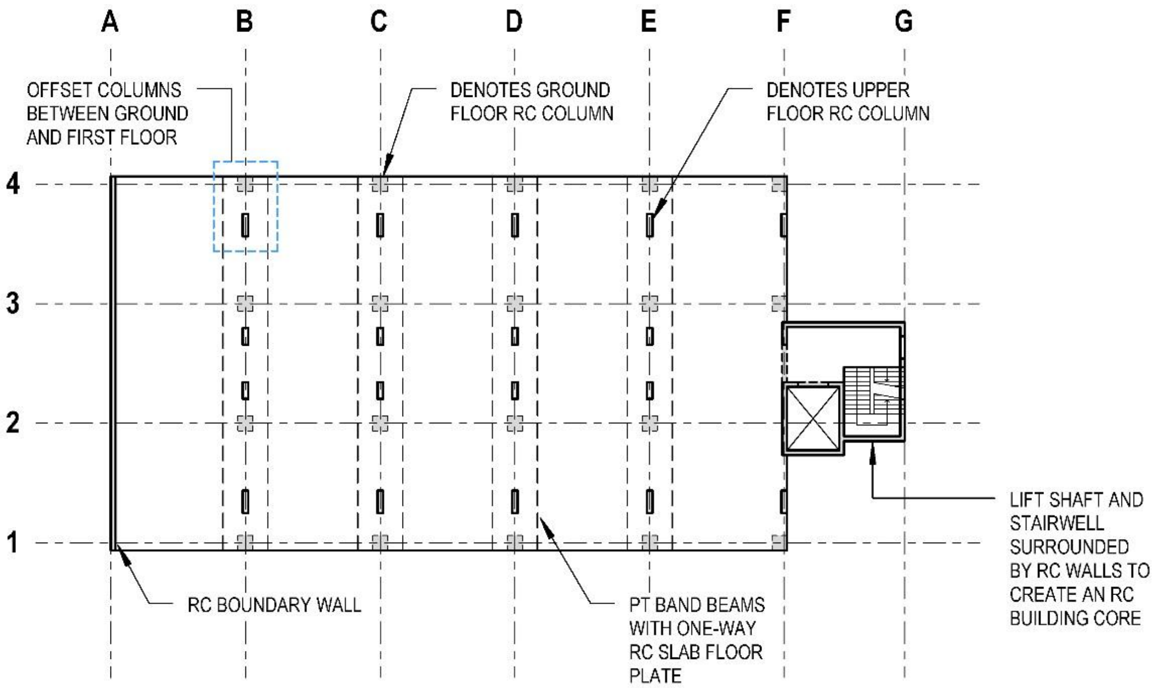

Figure 2. Category 2 building with RC walls as the dominant lateral load resisting systems.

Figure 3. Category 2 building with perimeter RC walls.

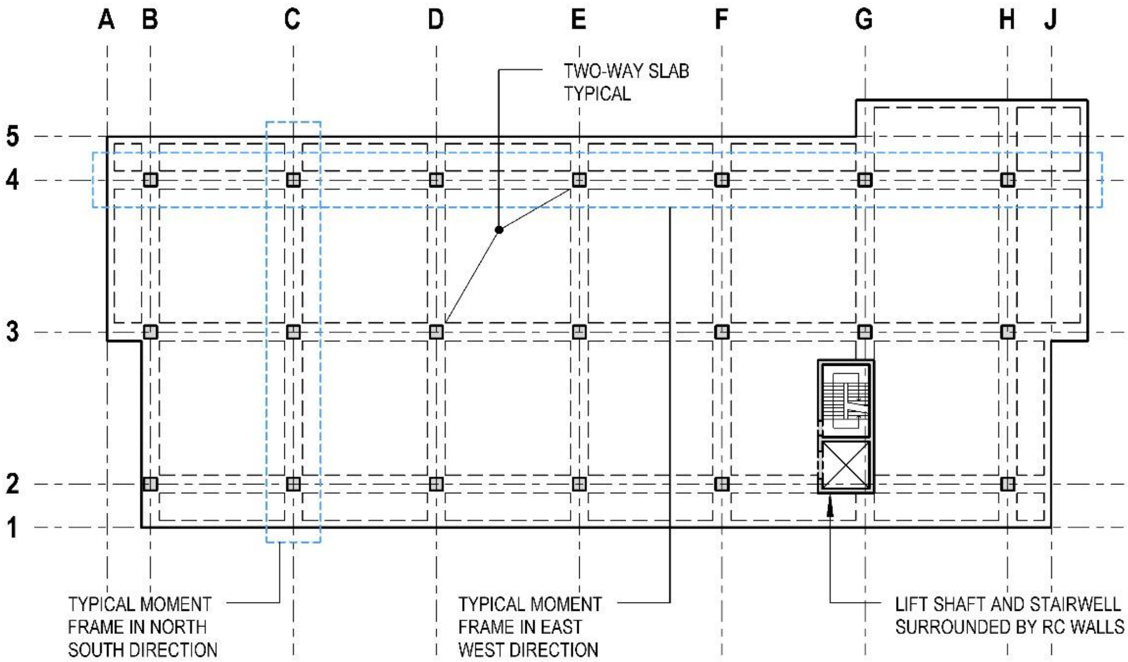

Figure 4. Category 3 building with dual RC moment resisting frame and RC walls.

Definition of Performance Levels and Damage States

Various documents have been published to define and describe performance levels and damage states. GEM guidelines (D'Ayala *et al*., 2015) provide a comprehensive review of relevant studies and recommendations made to date. Four structural damage states are typically defined, namely Slight, Moderate, Extensive and Complete, as in the HAZUS Technical Manual (FEMA, 2012). The definition and detailed description of each performance level or damage state have been consolidated from various seismic assessment guidelines (e.g., SEAOC, 1995; ATC, 2003; CEN, 2004) with the considerations of Australian conditions, as summarised in Table 1.

The most severe damage state (Level 4) of "Complete Structural Damage (*CSD*)". Technically, there could be excessive permanent lateral deformation or brittle failure of certain critical structural components, or loss of stability of part of the structure. The damage state of *CSD* in HAZUS is analogous to damage grades D4 (very heavy damage, partial collapse) and D5 (total or near total collapse) combined (Spence *et al.*, 2011) under the classification scheme used in the European Macroseismic Scale 1998 (EMS-98) (Grünthal, 1998). This level of damage is essentially consistent with the "Collapse Prevention" level as defined in the FEMA Publication 273 (ATC, 1997), or the "Near Collapse" level as in the Vision 2000 performance-based seismic design framework (SEAOC, 1995) and Eurocode 8 – Part 3, where the structural system is on the verge of experiencing partial or total collapse. Uncertainties surrounding the definition of "collapse" exist amongst the literature and practice. There are difficulties in accurately modelling the structural behaviour at the near-collapse state through structural analysis in practice, despite the advancement of contemporary modelling techniques.

Table 1. Proposed performance levels (or limit states) and damage states and the corresponding description of structural behaviour.

Performance Level	***Terminology***	***Description***	***F-Δ Behaviour***
1	Slight Damage, Immediate Occupancy, Operational, Serviceability, Cracking	Minimal damage may be observed with this performance level, however the damage and subsequent repairs should not affect the operational capacity of the facility. Hairline cracks are expected (i.e. small crack width). The structural response should be such that concrete compressive strains are within the elastic zone of the stress-strain curve and reinforcement tensile strains are associated with minimal inelastic behaviour.	Close to Linear Elastic
2	Moderate Damage, Repairable Damage, Damage Control, Damage Limitation, First-Yield	The structure has reached their yield capacity indicated by large cracks and minor spalling of cover concrete. The amount of damages is limited and the building is repairable. Limited inelastic behaviour is allowed in both concrete and reinforcement.	Effective Yield True Yield Strength
3	Extensive Damage, Significant Damage, Severe Damage, Life Safety, Spalling and Buckling, "Design" Ultimate (i.e. typical ULS in design standards, e.g. AS 3600)	The structure has reached its ultimate lateral strength capacity indicated by large cracks, spalled concrete and buckled main reinforcement. The building is non-repairable following the event. Very serious damage (and partial collapse of non-ductile elements) may have occurred but loss of life should be prevented. Inelastic behaviour are expected in both concrete and reinforcement.	Peak Lateral Strength
4	Complete Damage, Collapse Prevention, Near Collapse, Partial Collapse, "True" Ultimate	The building has low residual lateral strength and stiffness. There could be excessive permanent lateral deformation or brittle failure of certain critical structural components, or loss of stability of part of the structure. Parts of the structure has collapsed or are in imminent danger of collapse. The degree or proportion of collapse depends on the robustness of the structure and the properties of the construction materials.	Post-Peak Ultimate Drift

The various levels of section / member limit states, namely, cracking, first-yield, spalling and buckling, ultimate, as described in Priestley *et al.* (2007) are added into Table 1 for reference. From a design engineer or material standards perspective, the term "ultimate limit state (ULS)" refers to peak lateral strength that should be correlated to Level 3. Such limit state is considered "ultimate" based on a traditional force-based design approach, hence, such level can be described as a "design" ULS. However, in terms of the actual performance and response behaviours of structure, the "true" ULS occurs beyond the peak strength, which is governed by the ultimate drift capacity and is correlated to Level 4. There is debate to what point the ultimate drift capacity should be taken as (e.g. the point related to a 20 per cent drop in lateral strength), whilst the Priestley et al. (2007) definition is adopted in this study, which is the point before loss of axial load carrying capacity of the element ensues.

The associated inter-storey drift and material strain limits presented in Table 2 are proposed to be adopted in this study. Generally, damage of structures is governed by strain limits whilst damage of non-structural components is governed by drift limits. There are inevitable uncertainties in the predictions of structural demands as well as the material properties and structural capacity. Adequate conservatism should be provided for the determination of the drift and strain limit values.

Permanent drift limits are based on the analytical drift model proposed by Wibowo *et al.* (2014) and Wilson *et al.* (2015). Material strain limits have been selected with reference to the guidance provided in Priestley *et al.* (2007) and Sullivan *et al.* (2012) pending a thorough review of the recommended limits when the experimental testing program which is currently conducted by the authors has been completed. In the interim, the strain limits for steel at higher damage levels are selected on the conservative side. The recommendation of 0.6 times the ultimate strain in Priestley *et al.* (2007) based on low-cycle fatigue behaviour is considered not relevant to limited ductile structures in Australia. With the lighter amount of transverse reinforcement, buckling of longitudinal reinforcements is mainly prevented by the cover concrete. Hence, the complex mechanism that governs bar buckling is not likely controlled by the ultimate strain of the reinforcement.

Table 2. Proposed inter-storey drift and material strain limits associated to different performance levels and damage states.

Performance Level	***Terminology***	***Damage Factor****	***Transient Drift Limit (%)***	***Permanent Drift Limit (%)***	Concrete Strain ε_c	Steel Strain ε_s
1	Slight Damage	15% (< 17%)	0.5 (not specified) 0.4 (brittle NSC[#]) 0.7 (ductile NSC)	0.2	0.001	0.005
2	Moderate Damage	30% (10–33%)	1.5	0.5	0.002	0.01
3	Extensive Damage	100% (> 30%)	Varying with Axial Load Ratio, 2.5 if not specified	Varying with Axial Load Ratio, 1.0 if not specified	0.004	0.02
4	Complete Damage	100% (> 60%)	Depending on types of elements, and varying with axial load ratio		0.006	0.03

* Damage Factor (DF) is defined as the repair to replacement cost ratio. Structural and non-structural damages are included. A single value is recommended for each performance level, whilst the typical range as reported in the literature is given in the brackets for reference.

[#] NSC = non-structural components

A Damage Factor (DF) would need to be assigned to each damage state for undertaking economic loss assessment. DF is conveniently defined as ratio between the repair cost and the replacement cost. Both structural and non-structural damages should be included. It is found that the values of DF vary significantly in the literature, as reported in the GEM guidelines (D'Ayala *et al*., 2015). A range of values has been generalised for each damage state, as shown in the brackets in Table 2. For example, HAZUS recommends 2% for slight damage, 10% for moderate damage, 50% for extensive damage and 100% for complete damage.

As discussed in Crowley *et al.* (2005), the DF in some studies, including the recommendations in HAZUS, are likely to have underestimated real costs, should non-structural damages also be taken into account. Further, should the repair costs exceed 50% of the replacement cost (i.e. for extensive damage), the owner or insurer would prefer demolishing and re-constructing the building, as the expense would not be justified to invest into the repair of a heavily damaged building. Hence, a set of conservative values is recommended for the purpose of this study. The DF for Extensive Damage and Complete Damage are both 100%, with the consideration that buildings are not repairable (or not economical to repair) should it be subject to extensive damage, as defined in Table 1.

Seismic Hazard and Site Classification Scheme

The computation of annualised collapse probability requires seismic hazard predictions for annual frequency of exceedance as low as 10^{-5} or sometimes lower than 10^{-6}. The only set of hazard results that provides estimates for annual frequencies down to 2×10^{-5} (i.e. return period of 50,000 years) for Melbourne, Australia, can be found in Somerville *et al.* (2013), and is therefore adopted in the project pending further review when more updated hazard information becomes available.

A wide variety of near-surface geological conditions should be considered in the study. A site shall be characterised solely by the initial site natural period T_S of all the soil layers down to the depth of very stiff sedimentary materials, or bedrock. Classification of a site is based on the refined scheme as described in detail in Tsang *et al*. (2015) and summarised in Table 3, which has been recommended for incorporation into the Australian Standard for Earthquake Actions, AS 1170.4. It is noted that the upper boundary of 0.6 sec has already been used for Class C sites in the current edition of AS 1170.4 and has recently been evaluated and supported by Amirsardari *et al.* (2016).

There are minor changes to the description of the soil types in AS1170.4–2007, which become less ambiguous. For a site with $T_S < 0.15$ s where the soil layers are very thin and/or stiff, the site could be classified as a rock site. It is expected that the impact of earthquake hazards on a building which is located on rock sites should be minimal, in terms of both economic loss and casualty. Hence, rock site conditions can be ignored in this study.

Table 3. Proposed site classification scheme.

Site Class	Description	Site Period T_S (s)
A & B	Rock	$T_S < 0.15$
C	Stiff Soil	$0.15 \le T_S < 0.6$
D	Flexible Soil	$0.6 \le T_S < 0.9$
E	Very Flexible Soil	$0.9 \le T_S \le 1.2$
S	Special Soil	$T_S > 1.2$

In a recent study (Tsang *et al*., 2016) of annualised collapse risk assessment of soft-storey buildings with precast RC columns (Category 1 as defined earlier), the study buildings were assumed to be located on a suite of sites covering various soil conditions, with natural period of the whole soil layer, T_S, equal to 0.3 sec (Class C), 0.5 sec (Class C), 0.7 sec (Class D), 1.0 sec (Class E) and 1.2 sec (Class E). For simplicity, it is assumed that the weighted average shear wave velocity over all the soil layers of the five sites is 240 m/sec, and their total thickness, H_S, are as listed in Table 4.

Table 4. Total thickness of soil (H_S) and site natural period (T_S) of each site (weighted average shear wave velocity over the whole thickness is 240 m/sec for all five sites), and the corresponding site class based on the refined site classification scheme for AS 1170.4 (Tsang *et al.*, 2015).

Site	H_S (m)	T_S (sec)	Site Class
1	18	0.3	C
2	30	0.5	C
3	42	0.7	D
4	60	1.0	E
5	72	1.2	E

For simplicity, the design response spectra for various soil sites can be derived from a simulation-based model for estimating resonant-like amplification behaviour (Tsang *et al.*, 2006a, 2006b, 2012, 2017). This can provide estimates of non-linear spectral amplification factors and site period lengthening factors for different sites and at different levels of ground shaking. For sites with a weighted average shear wave velocity of 240 m/sec, and shaking level corresponding to a return period of 2,500 years (for Melbourne as an example), the factor for site period lengthening is estimated to be around 1.4 and the spectral amplification factor at the shifted site natural period to be around 3.5. Table 3 summarises the second corner period, T_2, and the *PDD* for the five sites with a return period of 2,500 years based on Melbourne conditions. Similar calculations can be undertaken for the whole range of return periods that are considered in this study.

Table 5. Site natural period (T_S), second corner period (T_2) and peak displacement demand (*PDD*) of the design response spectrum of each site with a return period of 2,500 years.

Site	T_S (sec)	T_2 (sec)	*PDD* (mm)
1	0.3	0.42	48
2	0.5	0.70	78
3	0.7	0.98	110
4	1.0	1.40	157
5	1.2	1.68	168

Ground Motion Inputs for Dynamic Analyses

For annualised risk assessment of buildings, fragility functions would need to be developed for each indicator building. Time history analyses will be applied in the form of incremental dynamic analysis (IDA) which involves the use of a suite of ground motion time histories that are representative of a wide range of intensity levels. A suite of hazard-consistent earthquake scenarios, in terms of magnitude-distance (M-R) pairs, will have to be identified for ground motion simulations. The adopted approach can incorporate more geological information (e.g., locations and geometry of major faults), and to cover for a wider return period range (instead of simply scaling up low-shaking recordings which could mis-represent the spectral contents of the earthquake ground motion in a projected scenario).

A proposed suite of ground motion inputs for a typical rock site in Melbourne has been produced using program GENQKE (Lam *et al.*, 2000, 2010) and the intraplate source model proposed by Atkinson (1993), which has been calibrated to the crustal properties of Melbourne based on the methodology as described in Lam *et al*. (2006). The suite of ground motions consists of fifteen different intensity levels ranging from a peak ground velocity (PGV) of 15 mm/s up to 800 mm/s, with four M-R combinations for each PGV level. When selecting the M-R combinations for each PGV level, consideration was given to major faults in the Greater Melbourne area. The governing faults were determined to be the Selwyn fault (M_{max} = 7.7, R = 60 km), Muckleford fault (M_{max} = 7.8, R = 120 km) and an undefined fault that had a maximum magnitude of 6.6 with a minimum distance of four kilometres (refer Figure 5). The maximum magnitude of 6.6 on an undefined fault was set with the consideration that a larger fault is not likely to be undefined in populated areas of Victoria. Hence, ground motions of higher return periods are controlled by random events on undefined faults that are close to the building. The fifteen PGV levels and four associated M-R combinations for the suite of ground motions is shown in Table 6.

The determination of the M-R combinations for individual selected PGV values for the study area of Melbourne could be based on correlations of PGV with magnitude and distance which has been derived by the authors from a parametric study (refer Figure 5). This process consists of (i) simulating multiple ground motions for numerous M-R combinations ranging from M = 5 to M = 8 and R = 4 km to R = 150 km, (ii) calculating the maximum average response spectrum velocity (RSV_{max}) for each M-R combination, and (iii) determining the PGV value for each M-R combination by dividing RSV_{max} by 1.84, which is the ratio of PGV to RSV_{max} in accordance with AS 1170.4 (Standards Australia, 2007). The earthquake hazard value or notional peak ground acceleration (PGA) denoted by k_pZ in AS 1170.4 (in units of g) can be determined by dividing the associated PGV (in units of mm/s) by 750 (Wilson and Lam, 2007).

Should site-specific acceleration time histories be warranted, site response analyses, using a computer program such as SHAKE (Ordonez, 2014), can be carried out for given site-specific soil profiles and non-linear dynamic properties of the soil for different site classes. At least three appropriate bedrock acceleration time histories should be adopted in the site-specific site response analyses.

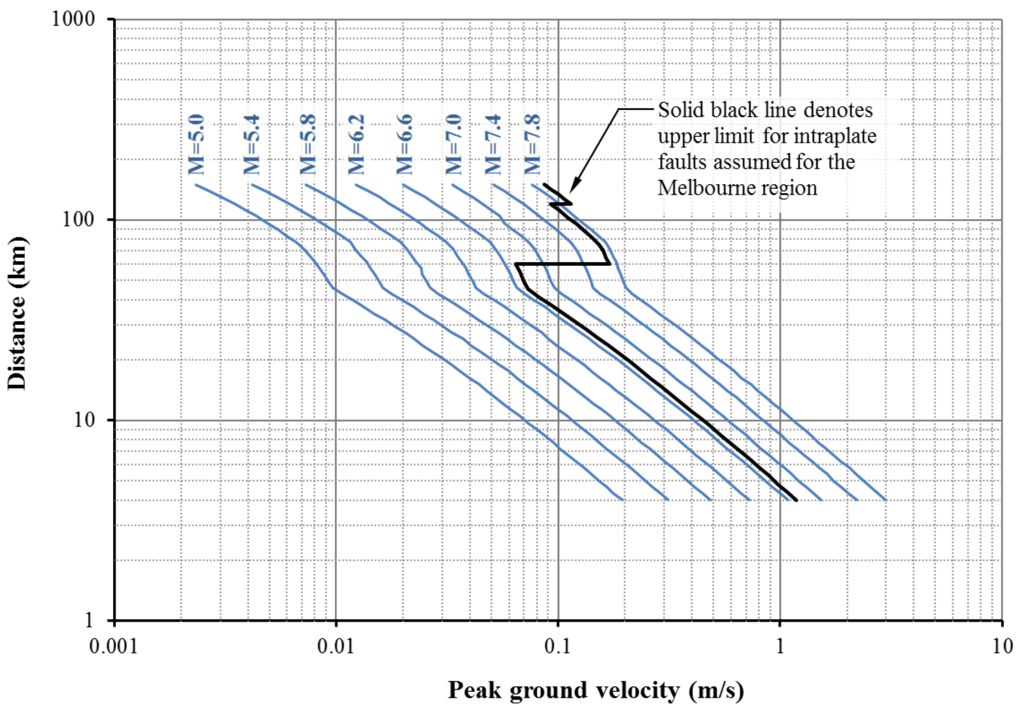

Figure 5. Peak ground velocity (PGV), magnitude (M) and distance (R) relationship for the Greater Melbourne Region.

Table 6. Earthquake scenarios for the Greater Melbourne Region.

Scenario	**PGV (mm/s)**	**M-R #1** M =	R =	**M-R #2** M =	R =	**M-R #3** M =	R =	**M-R #4** M =	R =
1	15	5	34 km	5.4	53 km	5.8	94 km	6.2	133 km
2	20	5	28 km	5.4	40 km	6	91 km	6.4	129 km
3	40	5	16 km	5.6	29 km	6.4	76 km	7	132 km
4	60	5	12 km	5.8	25 km	6.6	55 km	7.4	134 km
5	70	5	10 km	5.8	22 km	6.6	43 km	7.6	138 km
6	110	5	6.7 km	5.8	15 km	6.6	31 km	7.6	95 km
7	130	5	5.8 km	5.8	13 km	6.6	27 km	7.6	81 km
8	185	5	4.2 km	5.4	6.5 km	6.2	14 km	6.6	20 km
9	260	5.4	4.8 km	5.8	7.2 km	6.2	10 km	6.6	15 km
10	330	5.6	4.7 km	5.8	5.8 km	6.4	10 km	6.6	12 km
11	400	5.8	4.8 km	6	5.8 km	6.4	8.5 km	6.6	10 km
12	500	6	4.7 km	6.2	5.7 km	6.4	6.9 km	6.6	8.3 km
13	600	6.2	4.8 km	6.4	5.9 km	6.6	7 km	6.8	8.3 km
14	700	6.2	4.1 km	6.4	5 km	6.6	6.1 km	6.8	7.2 km
15	800	6.4	4.4 km	6.6	5.3 km	6.8	6.3 km	7	7.4 km

Note: shaded fields denotes M-R combination outside of fault scenarios assumed for the study area.

Conclusions

The framework and key parameters that are related to seismic vulnerability assessment of limited-ductile reinforced concrete (RC) buildings in Australia are presented in this paper for public review and comments by the Australian earthquake engineering community. The contents have been organised into four parts in this paper: (1) the three broad categories of vulnerable RC buildings; (2) performance levels or damage states; (3) seismic hazard and site classification scheme; and (4) hazard-consistent ground motion input. The outcome of the project will facilitate evidence-based informed decision making in relation to the need for seismic retrofitting, revision of codified design requirements and insurance policy.

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