

Prioritisation strategy for seismic retrofitting of reinforced concrete buildings in Australia

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ABSTRACT

This paper presents a study on the seismic evaluation and retrofit of limited ductile reinforced concrete (RC) buildings, which make up the bulk of built infrastructure in the central building districts and high-density residential areas in Australian cities. In low to moderate seismicity regions such as Australia, RC buildings will have structural elements that have been designed with limited to no ductility. The individual building needs to be assessed and ranked for their retrofitting priority. An assessment for retrofitting methodology involving a three-tiered approach will be introduced in this paper. The assessment framework including a tiered approach was developed to evaluate the potential vulnerability of Australian RC buildings and to facilitate decision making in relation to the need for seismic retrofitting. Structural threshold values related to the vulnerable features introduced in the framework were also investigated to support the identification process of the method. It is expected that the developed three-tiered methodology will provide a significant contribution to the seismic evaluation and retrofit of existing RC buildings in Australia.

Keywords: reinforced concrete building, seismic evaluation and retrofitting, tiered methodology

1. INTRODUCTION

Australia is entirely located within the Indo-Australasian plate with the records of the faults in a stable continental region. Seismic design and analysis of structures were not required before 1995. The buildings were commonly designed to carry gravitational and wind loads only. They have not been evaluated for their resistance against seismic actions (Clark et al., 2012; Lam & Wilson, 2008; Leonard, 2008; Menegon et al., 2017). As a result, many buildings that were built before 1995 may pose a big seismic risk to Australian society.

Reinforced concrete buildings with limited ductility make up the majority of the building stocks in Australian capital cities (Amirsardari et al., 2017). Most of the buildings in the regions that have been designed against the seismic actions are designed to only satisfy the non-collapse performance criterion under a rare 500-year return period earthquake and wind event. Due to the limited ductile detailing, RC frames and walls are vulnerable to brittle shear failure during a rare earthquake (Amirsardari et al., 2017; Ghobarah, 2000). The vulnerabilities of the RC buildings have been observed in many earthquakes, such as the Newcastle Earthquake in 1989 and Kalgoorlie Earthquake in 2010. The events gave warning to Australian society on the potential economic and social impacts of a large earthquake event. There is a necessity to assess the vulnerability of existing RC buildings in Australia.

Many existing methodologies such as hybrid vulnerability assessments (FEMA310, 1998; IITK-GSDMA, 2007), FEMA P-154 (FEMA154, 2015), Eurocode 8 (BSI, 2005), New Zealand Guidelines (NZSEE, 2014), modified Turkish method (Bommer et al., 2002), NRC guidelines (NRCC, 1993), IITK-GSDMA (IITK-GSDMA, 2007), ASCE/SEI 41 (ASCE/SEI41, 2014) and Japanese standard for seismic vulnerability assessment (JABDP, 1977) have been introduced during the last decades. These existing approaches on seismic evaluation, which are suitable for the different regions and countries, cannot be directly adopted without any modifications (Alam et al., 2012; ASCE/SEI41, 2014; El-Betar, 2018; Thermou & Pantazopoulou, 2011). Currently, Australia does not have a seismic evaluation and retrofitting standard, which is suitable for Australian buildings. It is important to develop a method that is easy to apply to identify vulnerable RC buildings in Australia for retrofitting priority. The method contributes to a large collaborative project which has the aim of developing cost-effective mitigation for building-related earthquake risk. This paper is going to address three key issues regarding to existing RC buildings in Australia: (i) common-modes of failure and vulnerable features for low ductility RC (LDRC) buildings, (ii) the methodology of three-tiered approach to vulnerable assessment; and (iii) structural threshold values related to the vulnerable features introduced in the framework to support the identification process of the method.

2. COMMON MODES OF FAILURE AND VULNERABLE FEATURES IN LDRC BUILDINGS

The vulnerability classification of LDRC buildings should consider not only the modes of failure of the buildings but also the construction form and the vulnerable features shown in Tables 1 and 2. The relevant literature informing the established threshold presented in Tables and 2 are summarised as follows:

- Torsional effects caused by asymmetrical structural plans will lead to additional displacements on the structural elements located at the edges of RC buildings, which will be investigated further in Section 4 (Lumantarna et al., 2016; Lumantarna et al., 2018; Lumantarna et al., 2017).

- Structural walls with a thickness that is less than 150 mm and contain centrally located reinforcements as a single layer of vertical rebar. These walls have been reported to display a buckling failure mechanism (AS3600, 2018; BSI, 2004; Rosso et al., 2016; Sritharan et al., 2014).
- Unseating damage of floor structures from their supports is quite common in constructions due to the difficulty of completely sealing the gap between the supporting structures and slabs. Another reason causing the unseating failure is the relative displacement of structure exceeding the available seat width under lateral load (Liberatore et al., 2013; Shrestha et al., 2015).
- Adjacent structures that do not have adequate separation may be subject to damage due to pounding action in an earthquake (AS1170.4, 2007; Eletrabi et al., 2010; Hao, 2015; Hao & Pearce, 2013). AS 1170.4 (2007) requires separations between adjacent buildings with more than 15m in height to a minimum of 1% of the building height to avoid seismic pounding. A study on the pounding effect in Australian conditions by Hao (2015) has found that separation of 1% of the taller building's height of adjacent buildings is sufficient to avoid damages due to pounding effects.
- Structural failure can occur in an earthquake due to the lack of structural load paths that can efficiently transfer structural loads to the foundation. The lack of a structural load path can potentially cause one structural member to be overloaded (ASCE/SEI41, 2014; NZSEE, 2018).
- Precast floors with hollow-core are prone to unseating failure due to inadequate seating or loss of seating (LOS) near the connection (Puranam et al., 2019). LOS can occur due to a trapped hollow core unit or spalling of concrete seating, as shown in Figure 1 (Jensen et al., 2007). An in-situ topping slab with or without reinforcement is often added on top of the pre-cast floor and connected to the beam through starter bars (Puranam et al., 2019). The prestressed forces within the precast diaphragm could cause floor mesh to fracture and floor toppings to crack (ASCE/SEI41, 2014). This is particularly an issue, if the mesh is the sole reinforcement for the floor system. The fracturing of the floor mesh could also cause separation of the perimeter concrete frames from the diaphragm (NZSEE, 2018).

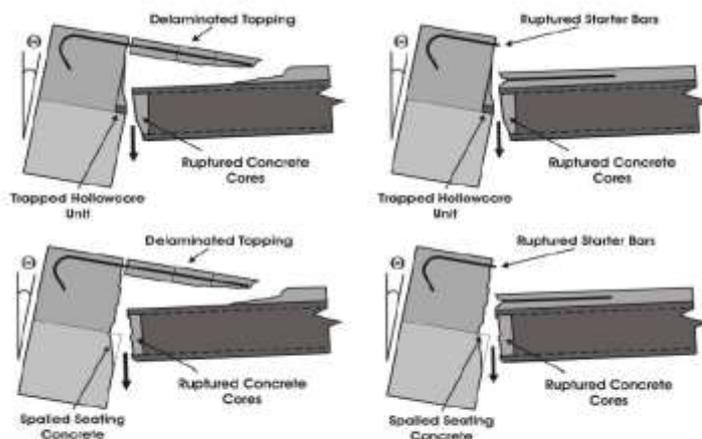


Figure 1 Possible failure mode due to the loss of seating (Jensen et al., 2007)

- Critical columns under high axial loads can fail due to excessive drift, which could lead to lateral load failure, axial load failure or buckling failure (AS3600, 2018; NZSEE, 2018; Raza et al., 2018; Truong et al., 2017). Columns with an

axial load ratio greater than 0.3 have been shown to display brittle behaviour under lateral loadings (AS 1170.4, 2007; Li et al., 2015; Wibowo et al., 2014b). Moreover, high strength concrete with concrete compressive strength higher than 50MPa is a brittle material requiring additional reinforcement detailing to avoid brittle failure (Al-Osta et al., 2018; AS3600, 2018; Bhayusukma & Tsai, 2014).

Table 1 Classification of vulnerable features A for RC buildings

Item	Vulnerable feature	Description	Category
1	Improperly braced building frame including frame with soft or weak-storey	An unstable building which does not contain structural walls to significantly contribute the building stability.	A
2	Fragile structural wall	The thickness of a structural wall is less than 150mm, which is generally consisting of only a single layer of longitudinal rebar located in the middle of the wall.	A
3	Unsecured or unfilled floor support	Lack of connection due to the improperly sealed gap between adjacent structural elements or limited seating width for supporting floor on adjoining structural wall or column.	A
4	Inadequate separation between buildings	The clear distance of setback from the boundary of the adjacent buildings that are more than 15m in height is less than 1% of the height of the taller building.	A
5	Lack of structural load path	The structure does not contain a complete, well-defined load path, which includes structural elements and connections that transfer the inertia forces associated with the foundation.	A
6	Geohazards including liquefaction issue, slope failure, and surface fault rupture	Liquefaction induced by an earthquake will happen when susceptible, saturated, loose, granular soil under the building is within the foundation soil at a depth of 15m. It could reduce the seismic performance of buildings. The building is not located sufficiently away to avoid potential earthquake-induced slope failure and rockfalls. Surface fault rupture is expected or anticipated at the building site.	A
7	Hollow-core floors	Precast floor with hollow-core floors and topping on the top	A

- Undersized columns with a high aspect ratio of more than 15 will display a buckling failure mechanism (AS3600, 2018; Lee et al., 2018). The limit of 15 was obtained from AS3600 (2018), which sets a limit for the slenderness ratio of RC columns to 120 to prevent buckling failure of columns. The limit for columns' dimensions depends on many factors such as concrete strength, spacing of the reinforcements, diameter of longitudinal and transverse reinforcement, total axial load on the column, aspect ratio (height/depth) of the column, and the inter-storey height of the buildings. Column size should be larger than 400 mm to avoid the congestion of longitudinal or transverse reinforcements in an RC column of high-rise RC buildings (Lee et al., 2018).
- Columns can display shear failure modes due to: i) low aspect ratio of the columns; ii) use of masonry infills; iii) captive or short column effect in a building (Guevara & Garcia, 2005; Wibowo et al., 2014a, 2014b; Wilson et al., 2009; Yi-An et al., 2014). Wilson et al. (2009) demonstrated that a column with an aspect ratio that is lower than 3.5 would display a shear failure mechanism. A follow-up study conducted by Wibowo et al. (2014b) set the limit to 4. Masonry infill walls that do not span over the full storey height can result in a

short column effect, making the columns prone to shear failure (Zhou et al., 2018). Moreover, adding masonry infill can increase the overall stiffness and strength of the buildings resulting in the reduction of inter-storey drift demand. However, the increase in stiffness also reduces the natural period of the structure and increase the maximum acceleration imposed on the structure. As a result, the lateral seismic load to the structure can be increased (Ko et al., 2014). The masonry infill is vulnerable in an earthquake, and its damage could lead to the development of soft-storey and shear failure (Magenes & Pampanin, 2004). Captive or short column effect can be identified if there are columns at a storey with a height/depth ratio is less than 50% of the normal height/depth ratio of the typical columns at the storey of the building (ASCE/SEI41, 2014). Shear demand on the captive or short columns will be increased as the moment demand on the columns remains constant (Loganathan & Shanmugasundaram, 2017).

- Nonductile reinforcement detailing can result in sudden brittle failure of RC buildings. Nonductile reinforcement detailing includes low percentage longitudinal and transverse reinforcement ratio, inadequate confinement in the boundary regions, inadequate and poorly designed transverse reinforcement for confinement and shear strength, strong beams, and weak columns (Amirsardari, 2018; El-Betar, 2018; Ghobarah, 2000). The reinforcement ratio of RC columns should not be less than 0.01 and more than 0.04 in the vertical direction. The minimum reinforcement ratio of RC columns is 0.0009 in the transverse (horizontal) direction (AS3600, 2018; Wibowo et al., 2014a). Likewise, the minimum reinforcement ratio should not be less than 0.0025 in the vertical and horizontal directions for an RC wall (AS3600, 2018; Goldsworthy & Gibson, 2012; Hout et al., 2018a; Hout et al., 2018b). When the structural drawings and reinforcement detailing are not available, existing buildings designed before 1995 can be assumed to have nonductile detailing.
- Vertical irregularity can be caused by setbacks, abrupt changes in strength, stiffness, geometry, or mass in one storey with respect to the adjacent stories (NZSEE, 2018). Transfer beams are vulnerable in an earthquake due to a high concentration of damage that could occur on the transfer beams, columns supporting the transfer beams and joints between the transfer beams and columns (NIST, 2016; Shahrooz & Moehle, 1990; Varadharajan et al., 2014). Moreover, vertical irregularities could also result in a soft or weak storey failure mechanism (Mwafy & Khalifa, 2017).
- Onerous site subsoil conditions of D and E classified as soft soil sites have a larger amplification of the soil response compared to site classes A, B, and C (AS3600, 2018). The larger amplification will lead to greater drift demand on the structural elements in the buildings (NZSEE, 2018).
- Horizontal irregularities can significantly increase the torsional effects and could result in excessive drift demand of buildings under earthquake excitation (ASCE/SEI41, 2014; Jereen et al., 2017).
- A mezzanine floor may be partially attached to the structural frames in a building and depend on the lateral load resisting elements of the building for its stability. The unbraced mezzanine floors could cause collapse of the whole building in an earthquake. Further, adding a mezzanine floor could also create a short or captive column effect, leading to a shear failure mechanism (ASCE/SEI41, 2014).

Table 2 Classification of vulnerable features B for RC buildings

Item	Vulnerable feature	Description	Category
1	High axial load on columns	High axial load on the column represents a high compression index in any column ≥ 0.3 , high strength concrete columns with concrete compressive strength (f_c') ≥ 50 MPa.	B
2	Undersized column	Undersized columns with an aspect ratio of more than 15 or dimensions less than 400mm.	B
3	Columns are prone to shear failure due to the low aspect ratio, use of brick infill, captive or short column effect	A column with an aspect ratio of less than four is considered shear critical. Brick infill walls built around the adjoining RC columns can result in shear failure of the columns. There are columns at a story with a height/depth ratio is less than 50% of the normal height/depth ratio of the typical columns at the story of the building	B
4	Non-ductile detailing	It includes features with non-ductile reinforcement and reinforcement content below the minimum reinforcement requirement (minimum longitudinal ratio 0.01 for columns and 0.0025 for walls, minimum transverse reinforcement ratio 0.0009 for columns and 0.0025 for walls) and lack of continuity/anchorage between the beam-column connection or slab or foundation. Strong beams and weak columns are also classified as non-ductile detailing.	B
5	Vertical irregularities	Vertical irregularities include discontinuities in the lateral load resisting systems or gravity load transferring path such as the application of transfer beam, and abrupt changes in stiffness, strength and mass between adjacent stories.	B
6	Onerous site subsoil conditions	Onerous subsoil condition with the maximum depth of soil above bedrock more than 40 m. The site with this kind of feature will be classified as class D and E site, according to AS1170.4-2007.	B
7	Horizontal irregularities	A structural plan features asymmetry due to asymmetrical locations of structural elements such as structural walls or core walls within the floor plan and irregularities in mass distribution and floor shape.	B
8	Mezzanine structure	There is an inadequate load path to transfer forces from the mezzanine to the main lateral load resisting system.	B
9	Deterioration of structural materials	There are clear signs of degradation of structural materials such as scaling, disintegration, erosion of reinforcement, delamination, spalling and cracking of the concrete	B
10	Inadequate wall anchorage and foundation dowels for the RC wall	Exterior concrete walls, which are relying on the diaphragm to provide the lateral support, are not anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Wall reinforcement is not dowelled into the foundation with vertical bars that are at least equal in size and spacing to the vertical wall reinforcement above the foundation.	B
11	Topping slab	A continuous reinforced concrete topping slab with thickness less than 65mm when it is connected to the precast concrete diaphragm and less than 75 mm when it is not connected to the precast concrete diaphragm.	B

- RC structures that have shown some deterioration such as scaling, disintegration, erosion of reinforcement, delamination, spalling, and cracking of the concrete can be considered vulnerable in an earthquake (ASCE/SEI31, 2003; Ma et al., 2018; Rodrigues, 2014; Zhang et al., 2014).
- Inadequate wall anchorage and inadequate foundation dowel can result in shear or tension and flexural failure mechanisms, respectively (ASCE/SEI41, 2014; NIST, 2016; NZSEE, 2018; Zhao et al., 2018). A floor diaphragm should be designed to have collectors that can transfer the in-plane shear force from the diaphragm to the vertical elements such as columns or walls. The collectors should extend from the vertical elements by an effective length, which is more than one development length of the reinforcement in tension (AS3600, 2018). A minimum number of anchors of two for each panel is required for securing the panel into the diaphragm. Precast wall panels should be connected to the building foundation. The absence of the connection between the precast wall panel and the building foundation will create discontinuity of the load path and reduce its ability to resist seismic forces (ASCE/SEI41, 2014). AS3600 (2018) requires a total area of longitudinal dowel reinforcement ($A_{st, \text{dowel}}$) that is not less than the total area of longitudinal tensile wall reinforcement ($A_{st, \text{wall}}$).
- Precast floors with thin topping slab can fail due to inadequate thickness to transfer shear forces between the precast elements (ASCE/SEI41, 2014; King, 1998; NZSEE, 2018). AS3600 (2018) sets a minimum thickness of 75mm for a continuous cast-in-place RC topping slab acting alone or 65mm for an RC topping slab with sufficient reinforcements connected to the precast floor. The topping slab at the roof or each floor must be connected or dowelled to the vertical elements such as shear or core walls to provide a complete load path to transfer shear forces to the vertical elements (ASCE/SEI41, 2014).

Based on the vulnerable features in LDRC building and the failure modes described above, the vulnerability of buildings in Australia can be classified into two categories A and B in Tables 1 and 2. A building is considered to be in a high priority for retrofitting if it possesses one vulnerable feature belonging to category A or more than one feature belonging to category B. The methodology is further described in Section 3.

3. THE METHODOLOGY OF THREE-TIERED APPROACH TO VULNERABLE ASSESSMENT

The methodology is subdivided into three levels of scan check. The purpose of a tiered approach is such that if a building passes level one, the building can be deemed safe, and it does not need to go through level 2 or level 3. Only a building that has not met level 1 and 2 checks needs to go through the level 3 check.

3.1 LEVEL 1 SCAN CHECK

Level 1 scan check is subdivided into two levels of scan checks, which are level 1.1 and 1.2 scan checks. Level 1.1 scan check is the first step to assess the vulnerability of the building. It is based on the overall height and characteristics of the building, which can be determined by inspection of the site or the design drawings if they are available. Likewise, it only involves a simple evaluation of the site and building, which does not contain any analytical or computation works. The acceptance criteria in level 1.1 scan check are listed in Table 3. Level 1.1 scan involves checking if the building has adequate lateral load resisting elements and the building features any one of category A vulnerable features presented in Table 1. A building can be deemed safe if the

building has adequate lateral load resisting elements that do not contain any feature of category A. Further check is not required.

Table 3 Acceptance criteria in level 1.1 scan check

Item	Building height range	Acceptance criteria
1	Building height up to 8m	The building is not containing any features belonging to category A.
2	Building height up from 8m to 50m	Buildings have adequate lateral bracings and does not have any vulnerable features which are classified into category A. Adequately braced buildings can include the following buildings. For example, a braced building with external structural walls from the foundation to the roof of the building is symmetrically designed on the floor plan. A symmetric building plan with a minimum of two major core walls of the same dimensions and a clear distance between two major core walls to be approximately equal to the width of the building can be classified as adequately braced building.
3	Building height more than 50m	Buildings have adequate lateral bracings and are free of any vulnerability features of category A.

When the assessment does not satisfy the acceptance criteria of level 1.1 scan check, level 1.2 scan check will be applied. Level 1.2 scan check involves identifying if there are any vulnerable features of category A in Table 1 and more than one vulnerable features of category B in Table 2. If the building did not contain any vulnerable features of category A and not more than one vulnerable feature of category B in Tables 1 and 2, it can be deemed to pass level 1.2 scan check. Otherwise, the level 2 scan check is needed.

3.2 LEVEL 2 SCAN CHECK

Level 2 scan check is performed to assess the torsional stiffness parameter and potential drift demand of the building caused by an earthquake. The process involves linear elastic analyses such as dynamic analyses or equivalent static analyses. The drift demand obtained for critical structural elements will be compared with the drift capacity to decide if retrofitting is required. The Generalised Force Method (GFM) developed in recent years by Lumantarna et al. (2018) can be used to replace the complex three-dimensional analyses. Furthermore, it can be used to estimate the displacement demand and the torsional stiffness parameter of the building. The method can be used in the level 2 scan check.

One of the purposes of seismic retrofitting is to increase the translational and torsional stiffness of asymmetrical RC buildings. Two parameters define the torsional response behaviour of asymmetrical buildings. They are torsional stiffness parameter (b_r) and eccentricity parameter (e_r), which are the torsional stiffness and eccentricity, respectively divided by the radius of gyration (r) of the building. Parametric studies were undertaken by Lumantarna et al. (2017) and Lumantarna et al. (2018). They reveal that the value of b_r to be less than 1.0 results in a high amplification of displacement demand of the building. Hence, it is proposed herein that buildings with values of b_r less than 1.0 should be deemed vulnerable.

The value of b_r can be calculated using equation (1).

$$b_r = \sqrt{\frac{e_r(e_r + B_r)}{\left(\frac{\delta}{\delta_{CR}} - 1\right)}} \quad (1)$$

Where B_r is the half-width of the building divided by the radius of gyration (r) of the building, δ is the effective deflection at the flexible edge when static lateral load is acting at the location of CM, δ_{CR} is the effective displacement corresponding to applying lateral load at the location of the CR which is the translational displacement of the building (Lumantarna et al., 2016; Lumantarna et al., 2018).

If b_r is less than 1, the RC building can be classified to have low torsional stiffness and should be retrofitted. If b_r is equal or larger than 1, the RC building can be classified to have adequate torsional stiffness. The maximum displacement on the critical elements can be calculated using linear dynamic analyses or the Generalised Force Method. Seismic retrofit is needed when the drift demand on the critical column exceeds the drift capacity. The drift capacity for a column can be calculated by applying the recommendation introduced by Raza et al. (2018). The equations to estimate the drift capacities of the column are listed in Equations (2) to (5) (Raza et al., 2018).

$$n = \frac{P}{A_g f_c'} \quad (2)$$

$$\delta_{af} = 5(1 - 2.0n) + \left(\rho_h \sqrt{\frac{f_{yh}}{f_c'}} \right) \quad (3)$$

$$(\delta_{lf})_{flexure} = 3(1 - 2n) + \left(\rho_h \sqrt{\frac{f_{yh}}{f_c'}} \right) \quad (4)$$

$$(\delta_{lf})_{shear} = 1.75(2.3\rho_h - n) + 0.8 \left(\frac{a}{h} - 1 \right) \quad (5)$$

Where, δ_{af} is the drift at axial load failure of column (%), $(\delta_{lf})_{flexure}$ is the drift at lateral load failure for flexure-critical columns (%), $(\delta_{lf})_{shear}$ is the drift at lateral load failure of shear-critical columns (%), n is the axial load ratio, ρ_h is the transverse reinforcement ratio by area (%), f_{yh} is the transverse reinforcement yield strength (MPa), and f_c' is the concrete compressive strength (MPa), a/h is aspect ratio, a is the shear span, h is the total depth of section.

If the drift demand on the critical structural element did not exceed the drift capacity of the element, the building could pass the level 2 scan check and be deemed safe. RC building that does not pass the level 2 scan check can be deemed unsafe. Consequently, retrofiting is recommended for the building. Alternatively, a level 3 scan check can be performed.

3.3 LEVEL 3 SCAN CHECK

Level 3 scan check is a more rigorous analysis based on non-linear behaviour of RC buildings to check the conservative results from the level 2 scan check if it is necessary. Level 3 scan check involves non-linear time-history analyses or capacity spectrum analyses.

4. DEFINITION OF ADEQUATELY BRACED BUILDINGS

This section presents parametric studies to verify the requirements for adequately braced buildings in Table 3. Nine building models were created to test the acceptance criteria for adequately braced buildings. The super-imposed dead and live load are respectively 1.5kPa and 4kPa. The live load on the roof is 0.25kPa. The buildings were assumed to be located on a site class D site in Melbourne. The key dimensions of the structural elements are presented in Table 4. Dynamic response spectral analyses were conducted using the program ETABS (CSI, 2015). GFM was applied to calculate the value of b_r . The base connections for columns and walls were assumed to be fixed for

the first three case studies in Table 4. When the fixed connections are applied at the base of the columns, the structural frame will contribute to the torsional stiffness of the buildings. The base connections for columns were assumed to be pinned for the rest of the case studies in Table 4. Hence, the contributions from the moment resisting frames (MRFs) to the torsional stiffness of the buildings are ignored.

The effects of the number of cores on the value of b_r and the displacement demand of asymmetrical buildings were investigated using two different building layouts shown in Figure 2. Results from dynamic analyses are presented in Figure 3. The displacement demand at the critical edge of the building is presented in the form of an edge displacement ratio (Δ/Δ_0) shown in figure 3b. It is the maximum displacement demand of the three-dimensional model of the buildings at the edge divided by the displacement demand of the equivalent two-dimensional model. For building layout 1, the contribution of MRFs to the value of b_r is also investigated. The contribution of MRFs to the value of b_r is shown to be significant. The effect of the number of cores on the value of b_r is less pronounced when the effect of MRFs is incorporated. It is a common design practice in Australia to ignore the contribution of MRFs to lateral stiffness and strength of the buildings and design the MRFs to carry the gravitational load. If the contribution from MRFs is ignored, buildings with two core walls that are closely spaced have b_r values that are less than 1.0. It is shown in Figure 3b that the edge displacement ratio can be much higher than 1.0 when b_r value is less than 1.0. Buildings with two cores that are closely spaced to act as lateral load resisting elements can be considered to be inadequately braced.

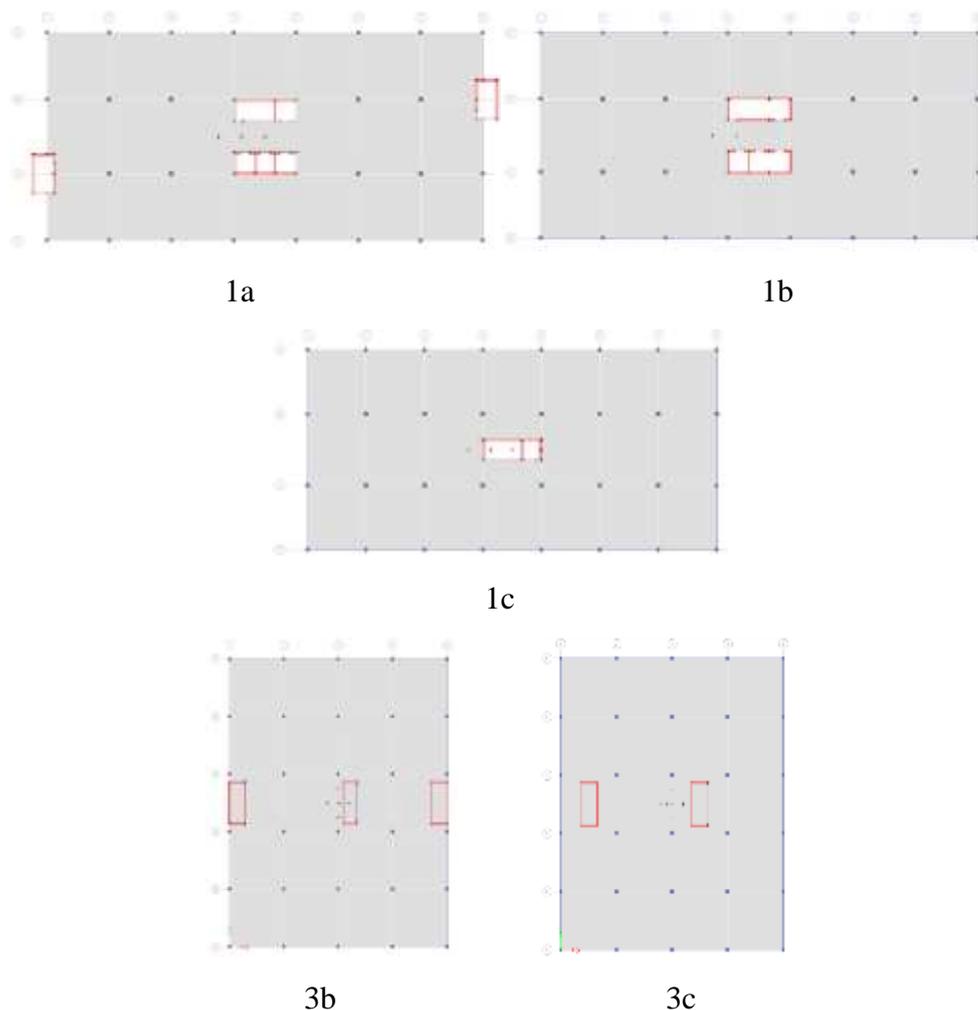
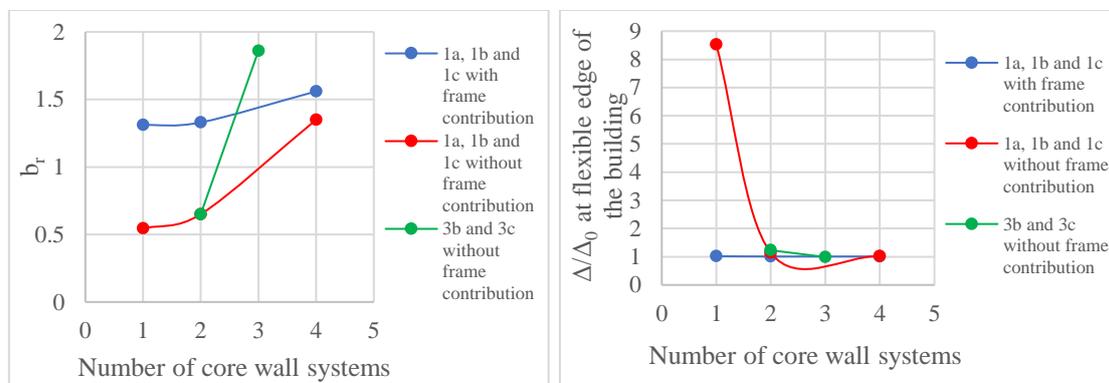


Figure 2 Layout of buildings used to investigate the effect of the number of cores

Table 4 Summary of the building geometries of case studies

Case study	Length (m)	Width (m)	Height (m)	Ext./Int column (m)	Beam	Slab thickness (mm)	No. of core wall	The thickness of wall (m)
1a	62.8	28.7	36.8	0.5x0.5/0.6x0.6	0.7Wx0.7D	0.25	4	0.25
1b	58.8	28.7	36.8	0.5x0.5/0.6x0.6	0.7Wx0.7D	0.25	2	0.25 0.2 0.15
1c	58.8	28.7	36.8	0.5x0.5/0.6x0.6	0.7Wx0.7D	0.25	1	0.25
2a	45	18	15	0.5x0.5	0.6Wx0.7D	0.22	1	0.2
3a	33.6	25.2	7	0.43x0.43	0.43Wx0.65D	0.18	2	0.2
3b	42	33.6	17.5	0.5x0.5	0.5Wx0.65D	0.18	3	0.2
3c	42	33.6	17.5	0.5x0.5	0.5Wx0.65D	0.18	2	0.2



(a) b_r

(b) edge displacement ratio Δ/Δ_0

Figure 3 The effect of the number of cores

The effect of thickness of the core walls on the values of b_r and the displacement demand of buildings are investigated using building layout 1b presented in Figure 2. The locations of the cores were also shifted along the one-axis of the building, which is shown in Figure 4. It is shown in Figure 5 that the value of b_r is not significantly affected by the thickness of the walls and the location of the cores. The edge displacement ratio in Figure 5b varies due to the effect of eccentricity on the maximum displacement demand of the building models. However, the difference in the edge displacement ratios between models 1b-1, 1b-2, and 1b-3 is small. It should be noted that the models incorporate the contribution from a moment-resisting frame (MRF). The value of b_r can be significantly lower, and the edge displacement ratio can be significantly higher if the contribution from MRF is ignored.

The effect of eccentricity within the building was investigated using building layouts presented in Figure 6. Within each building layout, the location of the central core is shifted along the horizontal axis. Results from the analyses in the form of b_r and Δ/Δ_0 values are presented in Figure 7. It is shown in Figure 7a the value of b_r is not significantly affected by the eccentricity. This observed trend is expected as the torsional stiffness is affected by the spacing between the cores more than their offset from the center of mass of the buildings. It is also shown that the maximum edge displacement is less sensitive to the changes in eccentricity when the value of b_r of the building is higher than 1.0. Figure 7b shows very high values of edge displacement ratio when the value of b_r is less than 1.0. This finding highlights the need for retrofitting to

increase the torsional stiffness of buildings when b_r value of the buildings is less than 1.0.

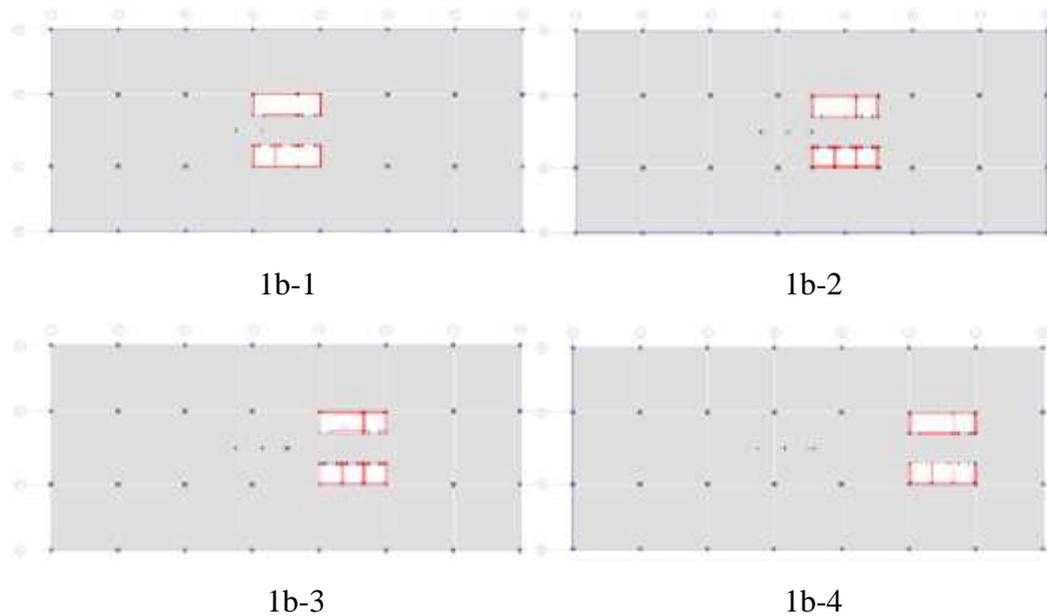


Figure 4 Layout of buildings used to investigate the effect of thickness of walls

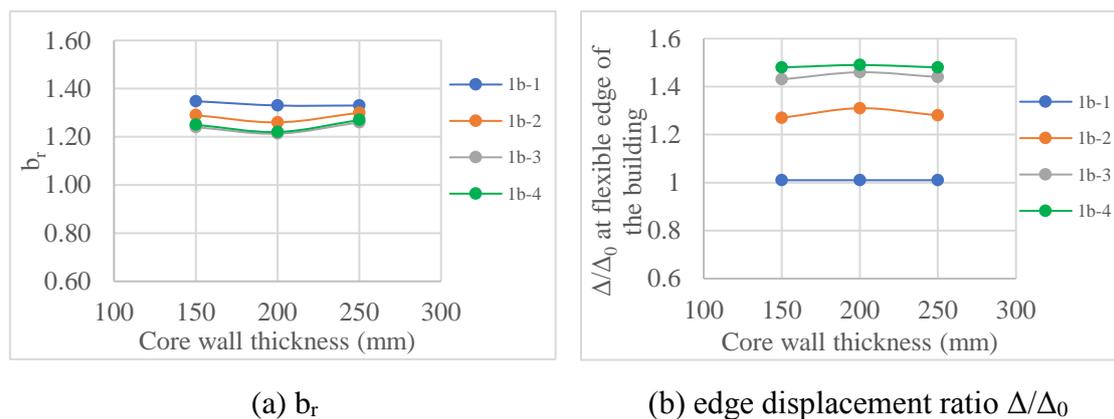


Figure 5 The effect of wall thickness

The effect of the spacing between the cores on the values of b_r and maximum displacement demand of buildings was investigated using the layout shown in Figure 8. The building cores presented in Figure 8 were shifted along the horizontal axis to introduce variations in the spacing between the cores. It is shown in Figure 9a that the value of b_r increases as the spacing between the cores increases. The value of b_r increases to a value greater than 1.0 when the spacing is approximately equal to the width of the building. The displacement demand of the building increases as the spacing between the cores increases, which is shown in Figure 9b. This observed trend is due to increasing eccentricity when the spacing between the cores is increased and unequal dimensions of the two cores shown in figure 9a. However, it is observed that the displacement demand does not increase indefinitely. This phenomenon is a result of the increase in the values of b_r as the spacing between the cores is increased. The first point of Figure 9b represents the displacement demand of the building model with closely spaced cores located at the center of mass and hence the eccentricity is close to 0, as shown in Figure 8.

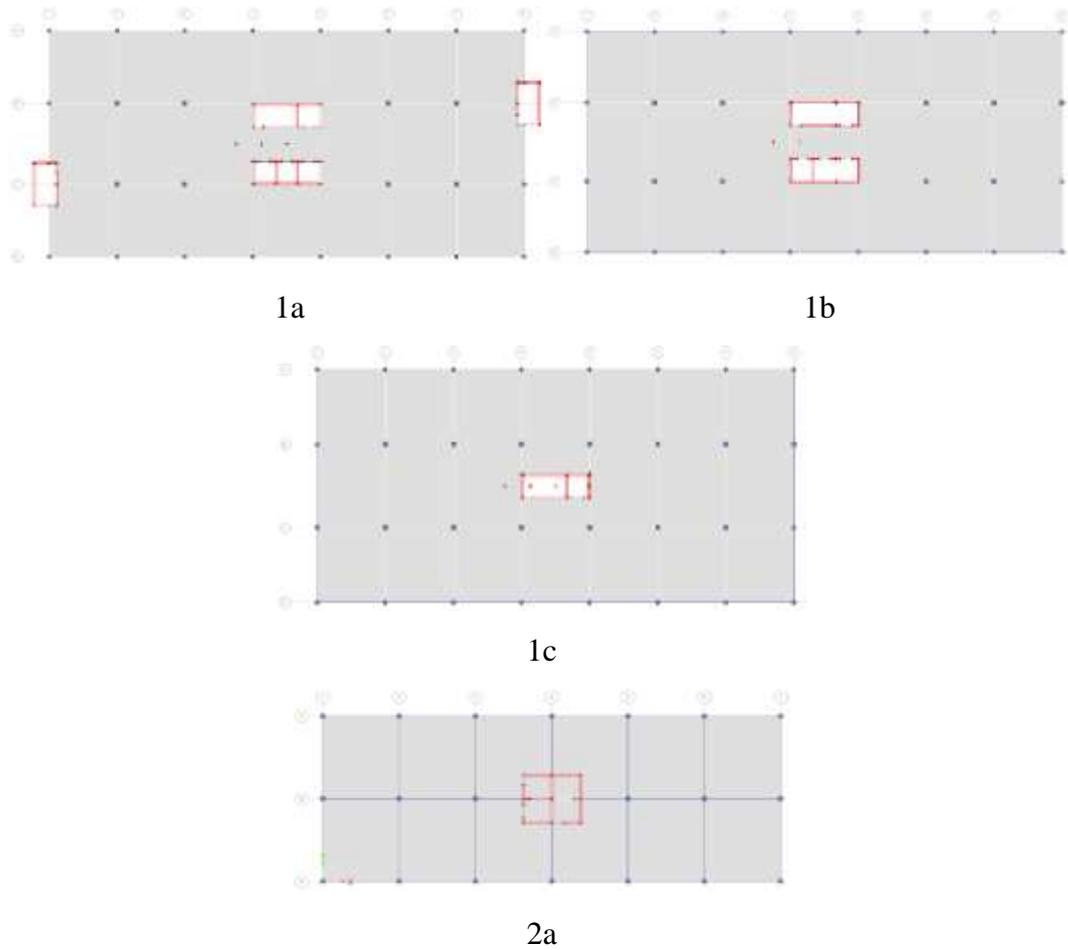


Figure 6 Layout of buildings used to investigate the effect of eccentricity

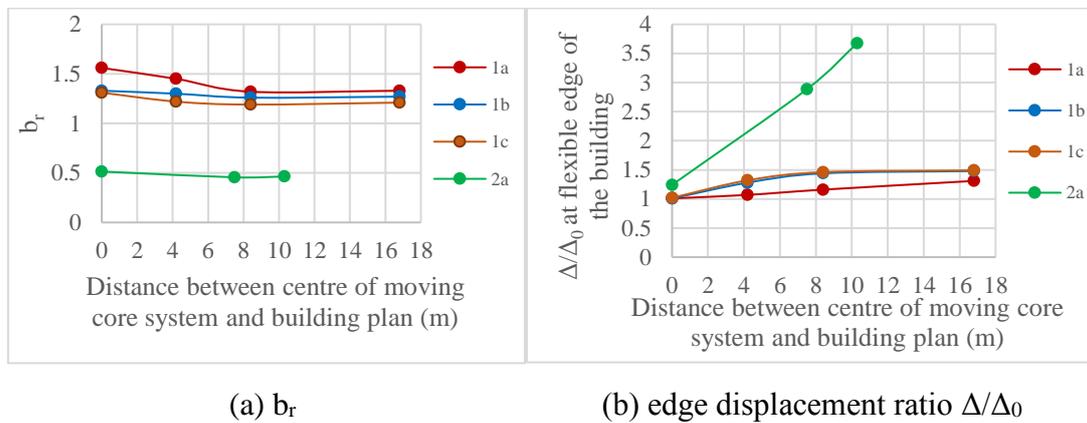


Figure 7 The effect of eccentricity

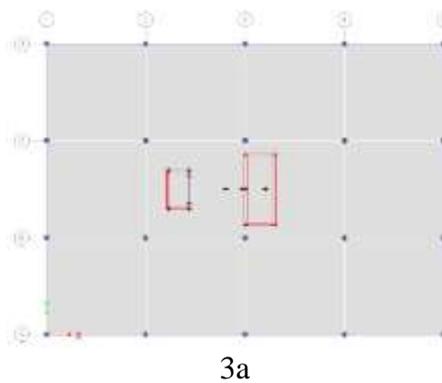


Figure 8 Layout of buildings used to investigate the effect of spacing between the cores

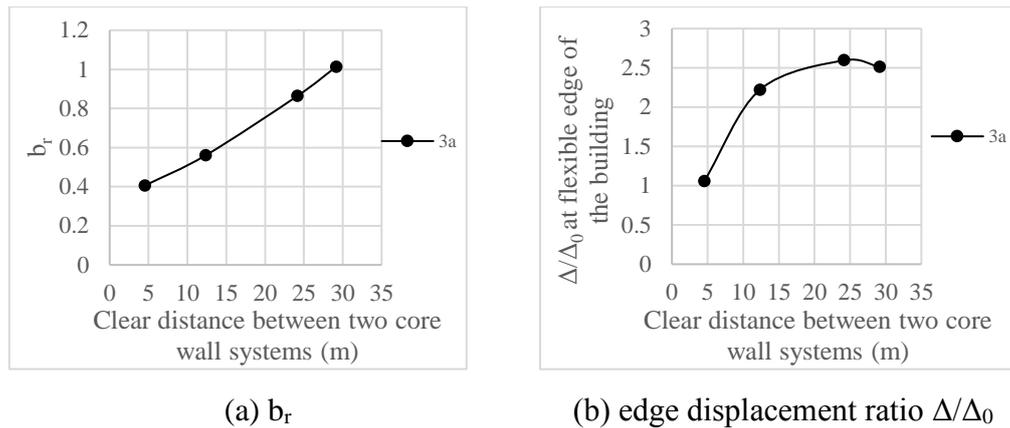


Figure 9 The effect of spacing between the cores

In summary, MRFs in a building could have a significant contribution to the torsional stiffness and b_r value of the building. As MRFs in multi-storey RC structures are only designed to transfer gravitational load, it is conservative to ignore the contribution from moment frames. The effect of MRFs on the displacement behaviour of asymmetrical RC buildings warrant further studies. It is shown from the parametric studies that building with a value of b_r less than 1.0 results in high amplification of displacement demands. Hence it is proposed herein that buildings with a value of b_r less than 1.0 should be deemed unsafe and requiring retrofitting. The results from the parametric studies have shown that a minimum number of 2 cores/shear walls of the same dimensions are required for a building to be classified as adequately braced. In addition, the clear distance between the cores should be approximately equal to the width of the building for the building to be classified as an adequately braced building.

5. CONCLUDING REMARKS

This paper presents an assessment methodology to prioritise the retrofitting of existing RC buildings. The methodology involves a three-tiered approach based on the identification of vulnerable features and is distinct from the methodologies which had been published in the literature.

A list of vulnerable features of RC buildings has been developed informed by the state-of-the-art review conducted by the authors. Further parametric studies on multi-storey buildings to verify the definition of adequately braced buildings provided in the developed methodology in Table 3. The tiered methodology introduced in this paper includes level 1 scan based on a visual check, level 2 scan based on linear elastic analysis using dynamic analyses or the Generalised Force Method, and level 3 scan based on non-linear dynamic or static analyses. The retrofitting requirement for a building can be identified through the identification of the torsional stiffness parameter, comparison between the drift capacity and drift demand of critical elements within the buildings. It is expected that the developed three-tiered methodology will provide significant time-saving in the vulnerability assessment of RC buildings in Australia, especially when many numbers of buildings need to be assessed.

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