

Development of Cost-effective Mitigation Strategy for Limited Ductile Reinforced Concrete Buildings

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Abstract

With the new awareness that most Australian reinforced concrete (RC) buildings have not been designed to withstand seismic actions, and are considered to have limited ductility, it is essential to consider retrofitting options. This study aims to evaluate the seismic performance of an archetypal Australian RC building, and then implement various retrofit techniques to find the most suitable retrofit. The structural performance of the buildings before and after various retrofitting were compared to study the effectiveness of the proposed methods as well as the effect on the seismic hazard design factor. SeismoStruct software was used to perform the nonlinear analysis of the structures. This paper contributes to a research with the overall aim of assessing the seismic performance of existing buildings and their expected failure modes, impact of retrofitting measures, and the associated costs.

Keywords: existing structures; seismic evaluation and retrofitting; limited ductility buildings; performance validation

1. INTRODUCTION

Since Australia is a region of low seismicity, seismic design of structures was not required, or even ignored. The seismic behaviour of limited ductility RC buildings have gained more attention, following their poor performance in the 1989 Newcastle earthquake. Due to the lack of historical perspective on seismic design, existing Australian reinforced concrete (RC) buildings can be extremely vulnerable and brittle. That is due to lack of adequate structural design and detailing for those buildings designed prior to the publishing of the earthquake loading standard in 1995. Several existing literature has touched on this topic in more detail, specifically for structures in Australia, such as Menegon (2018) and Amirsardari (2018). Seismic vulnerability assessment for a building that was deemed archetypal of Australian RC structures has been conducted by Amirsardari et al. (2018). This is the first step towards making calculated risk mitigation decisions regarding those structures. This current paper aims to further that research by proposing retrofit methods for the archetypal Australian buildings. Due to the prevalence of these structures, demolition might not be feasible nor economical. Herein the need for retrofitting options for these building types arises. This research was established with the Bushfire and Natural Hazards Cooperative Research Centre (BNHCRC) with the aim of assisting with risk mitigation decisions by providing practical retrofit solutions to the identified vulnerable buildings. The study aims to provide ready solutions in terms of retrofit strategies to avoid the need of analysing every similar structure in the future that requires retrofitting. This paper presents interim results of the study on limited ductile reinforced concrete buildings. Several retrofit options for a 2-storey limited ductile reinforced concrete building are presented. The building has been identified to be vulnerable in previous studies (Amirsardari, 2018). The retrofitting options have shown an improvement in the behaviour of the buildings. The retrofit options explored were simple with the purpose of being cost effective and easy to implement. This would not be applicable to buildings with higher importance levels, but rather the majority of the buildings which have been found to be limited ductile. In addition, it can be used as a preliminary study for the development of Australian seismic evaluation and retrofit standards of the existing buildings.

2. METHODOLOGY AND MODELLING

To determine the most appropriate retrofit method, a seismic assessment of the structure must first be undertaken. For this purpose, SeismoStruct software was utilised. SeismoStruct is a finite element software that is capable of predicting displacement behaviour of structures under static/dynamic loading, taking into account both geometric nonlinearities (global and local) and material inelasticity(SeismoSoft, 2018a). SeismoStruct allows the visualisation of the extent of damage under seismic events and excitations. It can also run both inelastic static pushover analyses and nonlinear dynamic time-history analyses. The program has been used in a range of previous research investigating the seismic performance of RC buildings (Almeida et al., 2016; Bolea, 2016; Carvalho et al., 2013; Houtl, (2017), Dias-Oliveira et al., 2016; Belejo et al., 2012).

For this paper, a nonlinear pushover analysis, applying triangular loads, was performed on the structures. This is because triangular loads simulate the earthquake loads better than uniform load applications. The buildings are pushed until collapse occurs, as this provides a better understanding of how the failure mechanism develops. As this study was carrying on from the work conducted by Amirsardari et al. (2018), it was decided that similar modelling techniques be adapted.

However, some adaptations had to be performed to allow those modelling methods to be implemented in SeismoStruct. To develop a deep understanding of how the software displays nonlinear behaviour, verification tests against experimental data of non-ductile reinforced concrete columns and walls and has been undertaken.

2.1. MATERIAL PROPERTIES

Both of the material properties chosen were previously recommended for use by Belejo et al. (2012) for a similar type of building modelling. For the concrete, the Mander et al. nonlinear concrete model - con_ma was utilized, also recommended for use by Amirsardai (2018). For the steel, the Menegotto-Pinto steel model - stl_mp was applied. The inputs for strain hardening parameter ($e_{sh}= 0.01$) and fracture buckling strain ($e_{su}= 0.05$) were utilized based on mean values for steel bars tested by Menegon et al (2015) and also utilized by Hoult (2017).

2.2. ELEMENT CLASSES

Element class deemed to be most suitable for this analysis is the infrmFBPH - fibre based plastic hinge model. This model features a distributed inelasticity displacement- and forced-based formulation but concentrating such inelasticity within a fixed length of the element. The advantages of this include reduced analysis time (since fibre integration is carried out for the two-member end section only), as well as full control/calibration of the plastic hinge length (or spread of inelasticity), which allows the overcoming of localisation issues. The number of section fibres used in equilibrium computations carried out at each of the element's integration sections also needs to be defined. The ideal number of section fibres, sufficient to guarantee an adequate reproduction of the stress-strain distribution across the element's cross-section, varies with the shape and material characteristics of the element cross-section. It also depends on the degree of inelasticity to which the element will be forced to. Automatic calculation of fibres was selected in for this model, in which 50 fibres are defined for a member's concrete area less than 0.1m^2 and 200 fibres for a member's concrete area more than 1m^2 . The number of fibres was obtained by linear interpolation for the in-between values. Each longitudinal reinforcement bar was assigned 1 additional fibre; added to the abovementioned number of fibres representing concrete elements(SeismoSoft, 2018b).

A plastic hinge length, in terms of percentage of wall/column height or beam length also needs to be specified. This is covered in the following section.

2.3. PLASTIC HINGE CALCULATION

The plastic hinge length was adopted from Priestley et al. (2007) with a minor adaptation as suggested by Hoult (2017).

The plastic hinge length as defined by Priestley et al. (2007) is expressed by:

$$L_p = kL_c + L_{SP} \quad \text{Equation 1}$$

Where,

$$k = 0.2 \left(\frac{f_u}{f_y} - 1 \right) \leq 0.08 \quad \text{Equation 2}$$

f_u = ultimate strength of steel

f_y = yield strength of steel

L_p = plastic hinge length
 L_{SP} = yield penetration length

$L_c = H$ = length from the base of the wall to the point of contra flexure (Cantilever height)

$$L_{sp} = 0.022f_y d_b \quad \text{Equation 3}$$

d_b = bar diameter of wall/column/beam

The adaptations based on Hoult (2017) include adding an additional term to allow for effects of tension shifts ($0.1l_w$), as well as considering the effective height ($L_c = 0.7H$). Equation 1 becomes

$$L_p = kL_c + 0.1l_w + L_{SP} \quad \text{Equation 4}$$

Where,

$L_c = h_e = 0.7H$ = effective height of the cantilever wall

l_w = length of the wall

The plastic hinge length equation was used to calculate all the lengths for the columns, walls, and beams. Those elements are verified in Section 3.

2.4. WALL MODELING

The walls have been modelled using the Wide-Column Model (WCM) proposed by Beyer et al. (2008). This method utilizes modeling each planar component of the wall with an individual line element, assigned to rectangular-fibre wall section. Then, these individual components are joined together using horizontal links. It is advisable to apply structural nodes at the corners of the wall so that all the nodes can be joined together by the links, which are to be applied at every half storey height. The benefits of this method, highlighted by Beyer et al. (2008) include:

- Modeling the distribution of shear forces between web and flanges accurately
- Inherent modeling of torsional stiffness of walls
- Allows monitoring of sectional forces acting on individual components of walls, thus assessing the likelihood of shear strength failure.

Note that the images in Figure 2 are that of a C section, however, this method has been applied for other wall shapes, such as the rectangular and block shape.

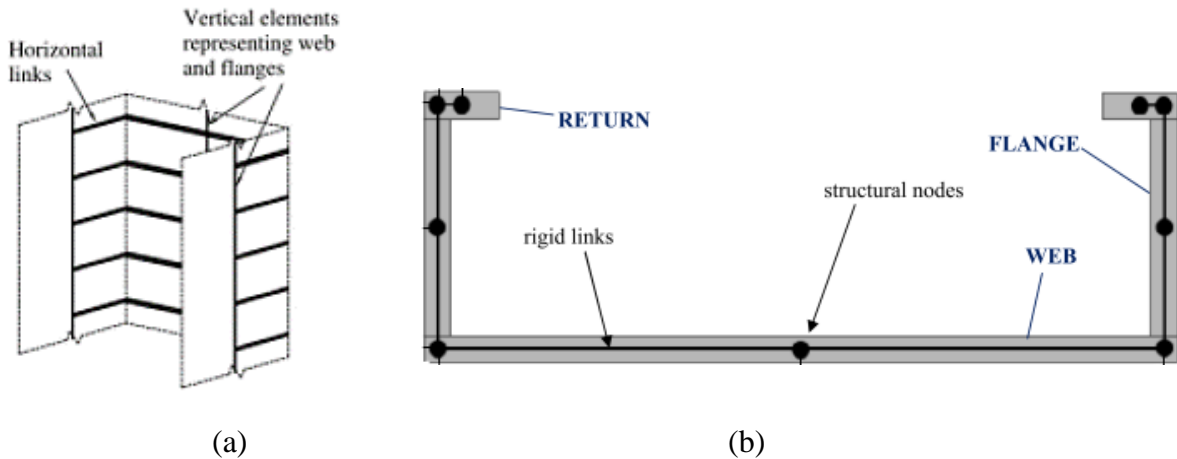


Figure 2: (a) Wide Column Model (Beyer et al., 2008) (b) Rigid Horizontal links on C shaped wall (Hoult, 2017)

3. MODELLING VALIDATION

To ensure that the proposed modelling methodology model the limited ductile and inelastic behaviour, it was verified against experimental data of different wall and column specimens, sharing the same inelastic behaviour that was expected of these existing Australian buildings. The comparison between the experimental results and the results from analyses using SeismoStruct are shown in Figures 2-9. Most of them display a reasonably accurate match in terms of backbone curve and maximum force reached, as well as displacement at collapse. It can be seen from Figures 2-9 that the degradation has also been captured quite accurately. The software can be slightly more conservative at times in terms of the maximum force reached as seen in Figures 4 and 5. The validation for the response of interconnected core walls from Figure 9 provided good accuracy in terms of both base shear and displacement. The individual response of the C-shape core wall has slightly more discrepancy, however, the behaviour is still acceptable since the interconnected model of both stair cores and C-shaped core is very similar to the Opensees data. Overall, these results are a good match and are acceptable in terms of modelling techniques.

3.1. COLUMNS

The design properties of the reinforced concrete columns that were used for the validation are shown in Table 1.

Table 1: Summary of design properties for verification columns

Column ID								Longitudinal		Transverse	
		b	h	L	N	f'_c	f_y	#bars	d	d	s
		<i>(mm)</i>		<i>(m)</i>	<i>(kN)</i>	<i>(MPa)</i>			<i>(mm)</i>		
Bousias et al. (2006)	D2-1	400	400	1.6	994	23.9	573	8	20	8	75
Lynn et al. (1996)	3CLH18	457	457	1.47	503	26.9	331	8	32	9.5	457.2
Raza et al. (2018)	S1	250	300	2.55	844	65	565	6	16	10	150
	S2				1485						
Takemura and Kawashima (1997)	1	400	400	1.25	157	35.9	363	20	12.7	6	70
	2					35.7					

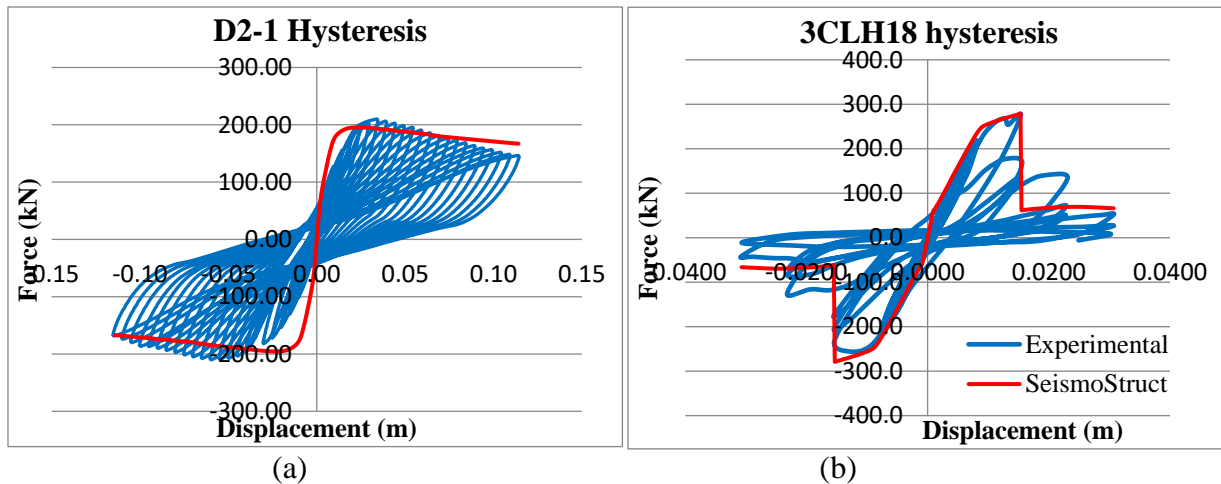


Figure 2: Hysteresis Loop Experimental data versus SeismoStruct results (a) D2-1 (Bousisas et al, 2006) (b) 3CLH18 (Lynn et al, 1996)

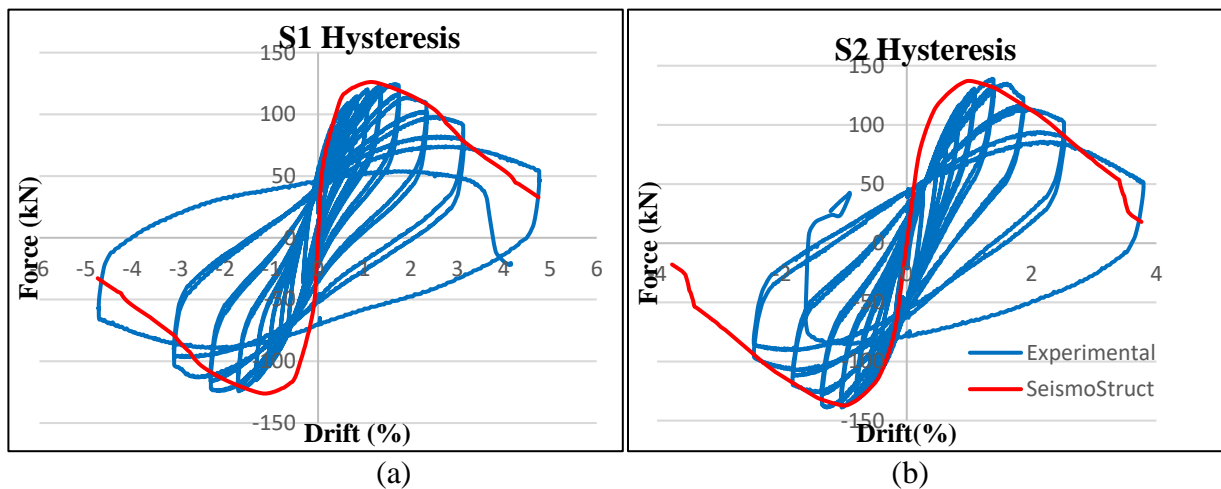


Figure 3: Hysteresis Loop Experimental data (Raza et al, 2018) versus SeismoStruct results (a) S1 (b) S2

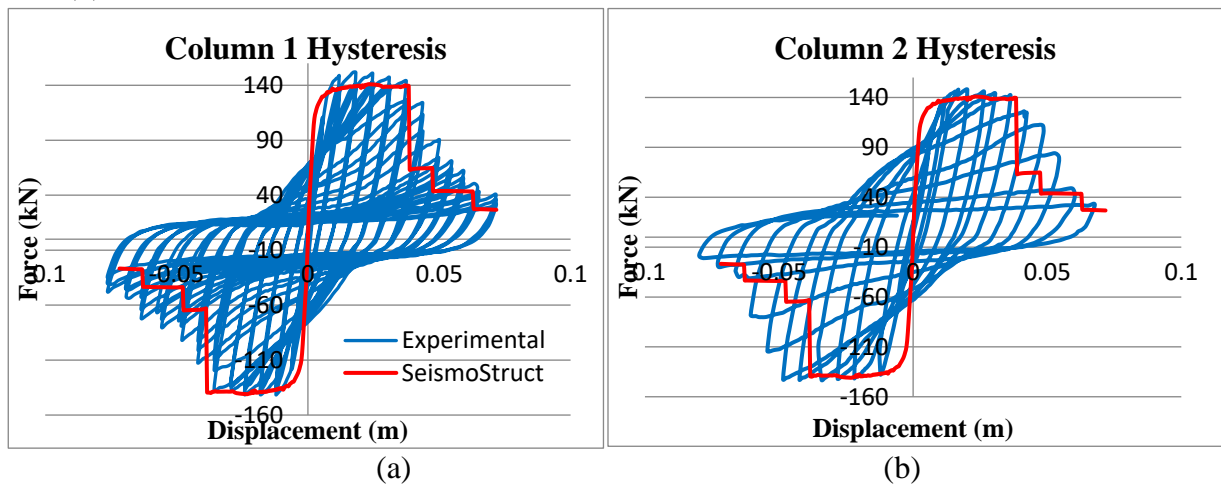


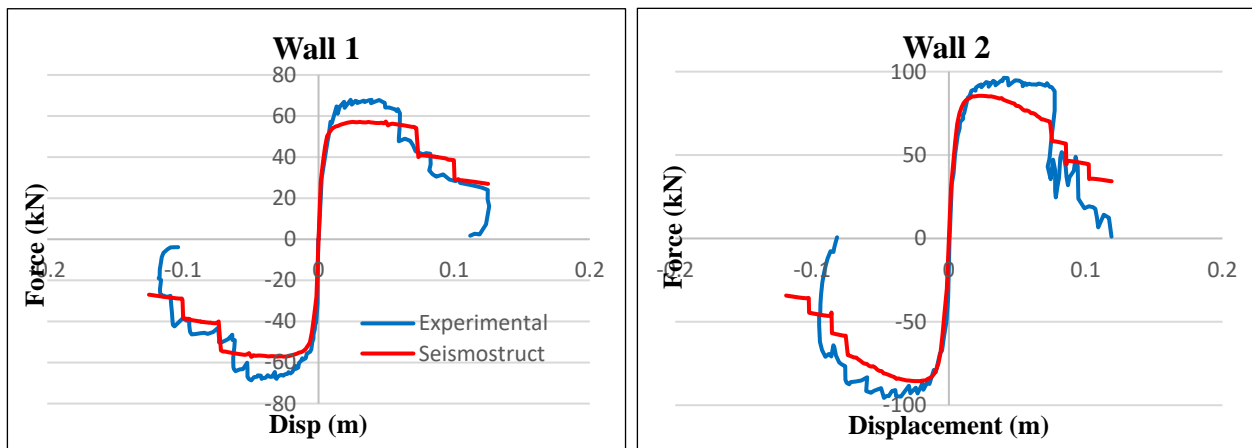
Figure 4: Hysteresis Loop Experimental data (Takemura and Kawashima, 1997) versus SeismoStruct results (a) 1 (b) 2

3.2. WALLS

Table 2 shows the design properties of the walls that were used for validation. Note that all walls are rectangular strips except for S02, Menegon (2018), which was a core wall.

Table 2: Summary of design properties for verification walls.

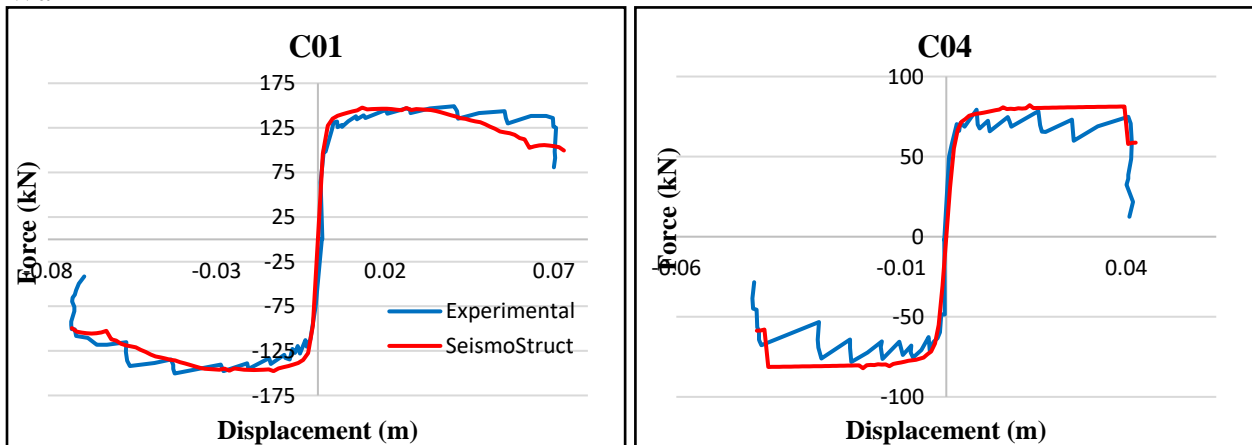
Wall ID		b (mm)	l (m)	H (m)	N (kN)	f'_c (MPa)	f_y	Longitudinal	Transverse		
								# of bars	d	d	S
								(mm)			
Altheeb (2016)	Wall1	120	0.9	2.75	190	35.2	500	5	10	10	200
	Wall2				187	34.7		10			
Lu et al. (2016)	C01	150	1.4	2.8	283	38.5	300	14	10	6	150
	C04				0	34.7					
Menegon (2018)	S01	400	1.2	2.6	585	41.9	532	14	20	12	250
	S02	1200			1200	31.6	544	64	12	10	



(a)

(b)

Figure 5: Backbone Experimental data (Altheeb ,2016) versus SeismoStruct results (a) Wall 1(b) Wall 2



(a)

(b)

Figure 6: Backbone Experimental data (Lu et al. ,2016) versus SeismoStruct results (a) C01(b) C04

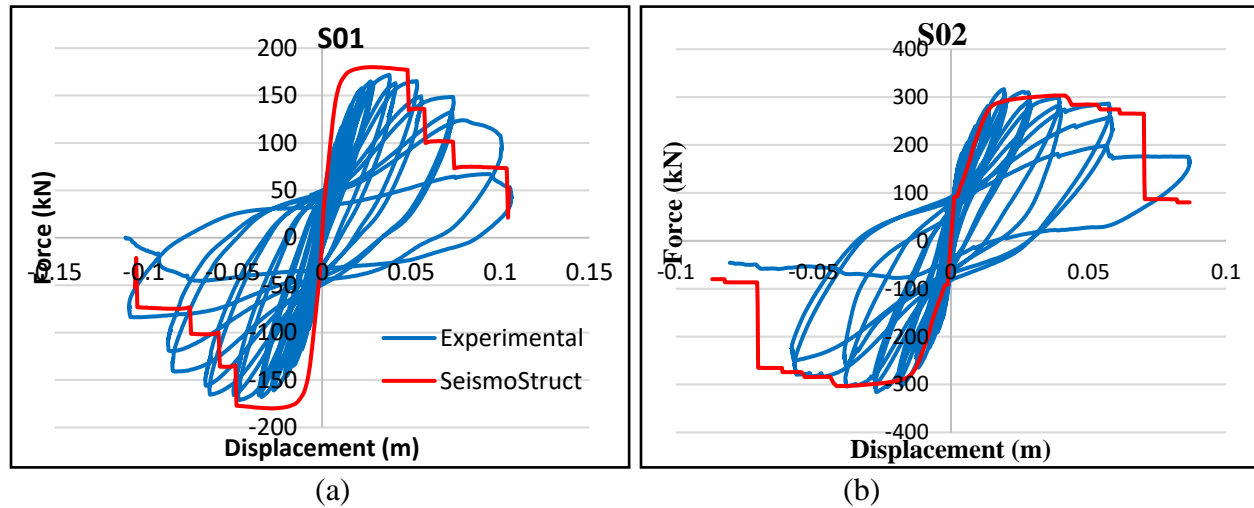


Figure 7: Hysteresis Experimental data (Menegon. ,2018) versus SeismoStruct results (a) S01(b) S02

3.3. CORE WALLS

To further verify the software, individual core walls and interconnected core walls were modeled and verified against the same walls modeled in Opensees under pushover analysis by Amirsardari (2018). The cores are that of a 5-story limited ductility RC building with 3.6m Interstorey Height. The properties and detailing of the walls can be seen in Table 3 and Figure 8. Note that the solid lines represent data from the literature (Amirsardari) while the dashed lines represent SeismoStruct results.

Table 3: Summary of design properties for verification walls.

	f'_c (MPa)	f_y (MPa)	$d_{\text{longitudinal}}$ (mm)	$d_{\text{transverse}}$ (mm)
Core walls	40	400	12	12

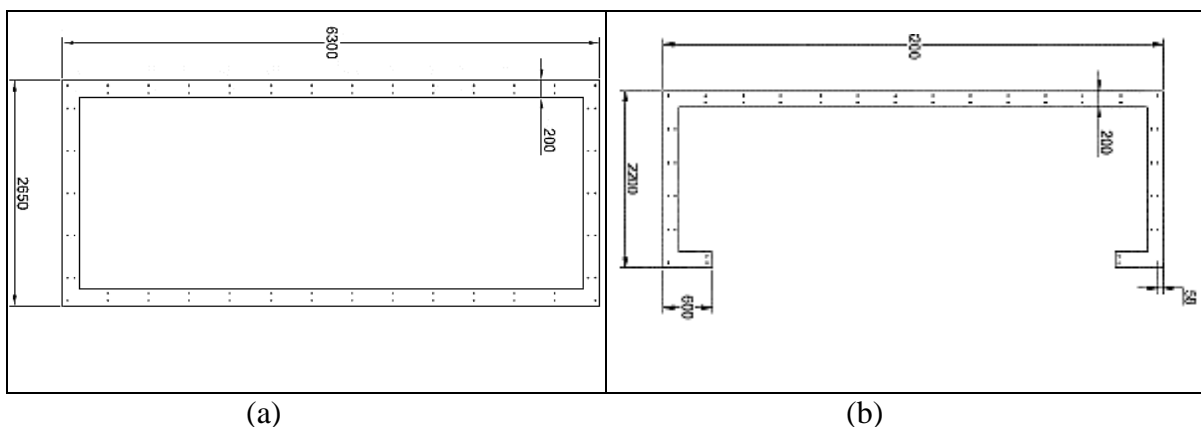


Figure 8: Typical design and detailing of (a) Stair Core (b) Lift Core (Amirsardari,2018)

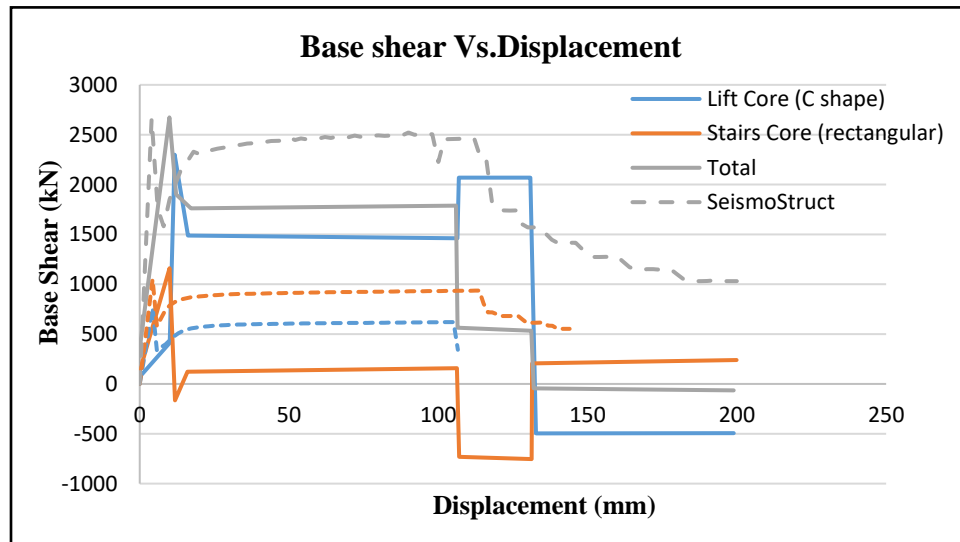


Figure 9: Response of Individual and Interconnected Core walls under pushover analysis modeled in SeismoStruct(dashed lines) and compared against literature (Amirsardari,2018)

4. Full Building Model

The proposed building is an archetypal building of RC buildings in Australia constructed prior to 1995. The seismic vulnerability of the buildings has been previously investigated by Amirsardari (2018). This building has been designed and detailed in accordance with AS3600:1988, as this was prior to the requirement for seismic load and design was mandated. It is a 2-storey building, with a 3.6 m storey height. The structural system constitutes of both movement resisting frames and shear walls.

The frames are ordinary moment resistant frames with detailing deficiencies such as:

- Inadequate transverse reinforcement (lack of confinement)
- Poor anchorage and splices of longitudinal bars in beams and columns
- Column bending moment capacity is close to that of beams, resulting in weak column-strong beam scenario

The interior gravity system was not modelled since it was expected that the perimeter frames fail prior to the interior gravity system.

The core walls have low longitudinal reinforcement ratios and poor anchorage, with no confinement. The material properties of the structural elements are presented in Table 4. The dimensions and details of the structural elements are presented in Figures 10 and 11. The plan view of the building is shown in Figure 9 a, while the three-dimensional building model developed in Seismo-struct in 12b. The walls and columns are fixed to the ground, and a rigid diaphragm assumption was adopted.

Table 4: Design Properties for building Elements

	Slab	Beams	Columns	Core walls
f'_c (MPa)	25	25	40	40
f_y (MPa)	400	400	400	400
$d_{\text{longitudinal}}$ (mm)	16	28	36	12
$d_{\text{transverse}}$ (mm)	12	12	10	12

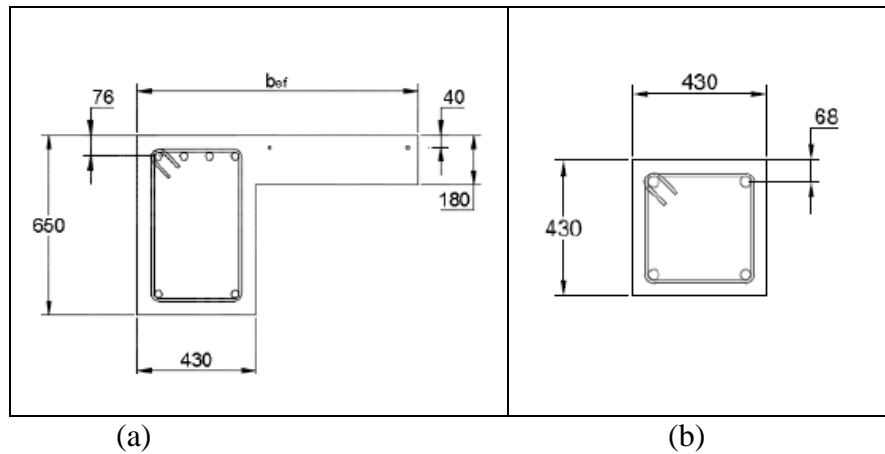


Figure 10: Typical design and detailing of (a) Beam (b) Column (Amirsardari,2018)

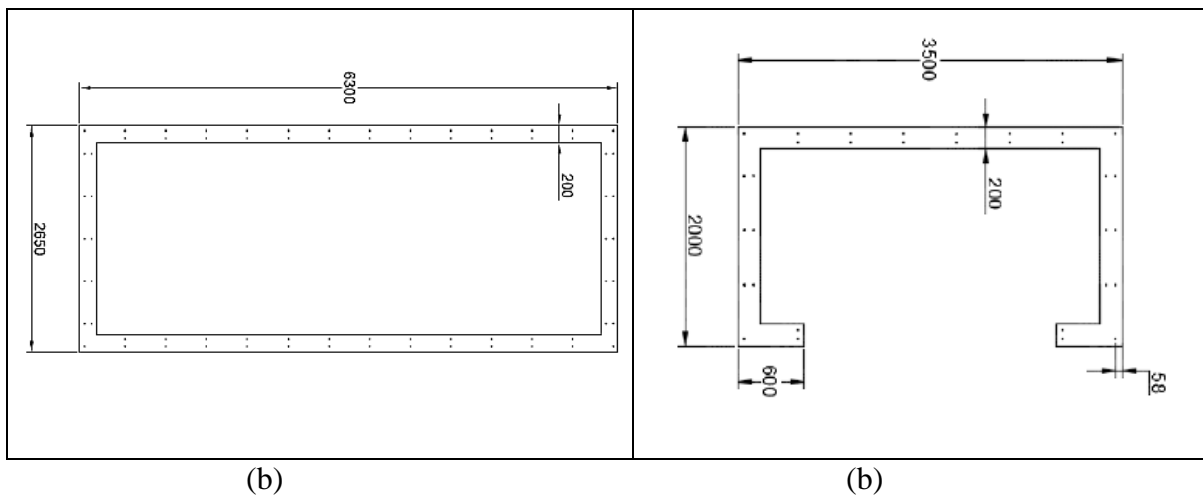


Figure 11: Typical design and detailing of (a) Stair Core (b) Lift Core (Amirsardari,2018)

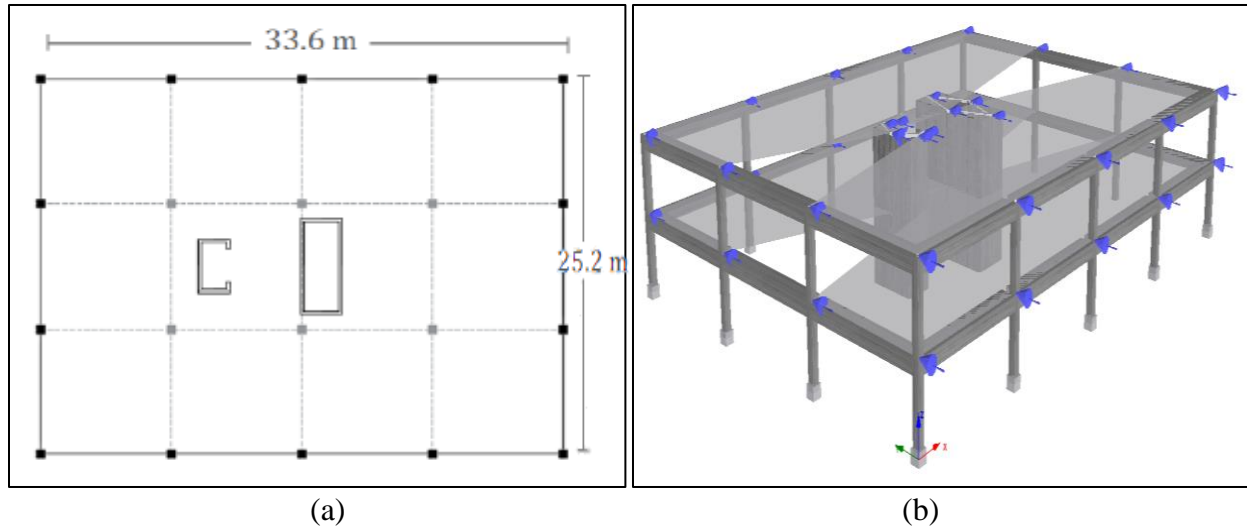


Figure 12: 2-Storey Limited Ductility RC building (Amirsardari,2018) (a) Layout (b) 3-D Model with blue arrows representing pushover loads on the structure

4. PROPOSED RETROFIT OPTIONS

The options explored in this paper are selected due to simplicity and ability of application with minimal intrusion and disruption to the operations of the building. The effectiveness of these following methods has been proven by several other studies such as Sunil and Sujith (2017), Huang et al. (2007), Caterino, N. & Cosenza, E. (2018), Tankut et al. (2006), Saatcioglu (2006) and Hussain et al. (2016). However, very few of these papers were exploring the effect on limited ductility buildings specifically. In this paper, the aim was to study the efficiency of the proposed methods on limited ductility buildings with typical Australian deficiencies.

5.1. X-Bracings

A circular hollow section was selected for the bracing members, with an external diameter of 100mm and 20mm section thickness. It was designed to have 500 MPa yield strength, and it was modelled using a truss element class with bilinear steel model on SeismoStruct, as recommended by the SeismoStruct Verification Report (SeismoSoft, 2018c). Four different Bracing layouts were investigated, however only 2 are presented, due to similarity in results, as shown in Figure 13.

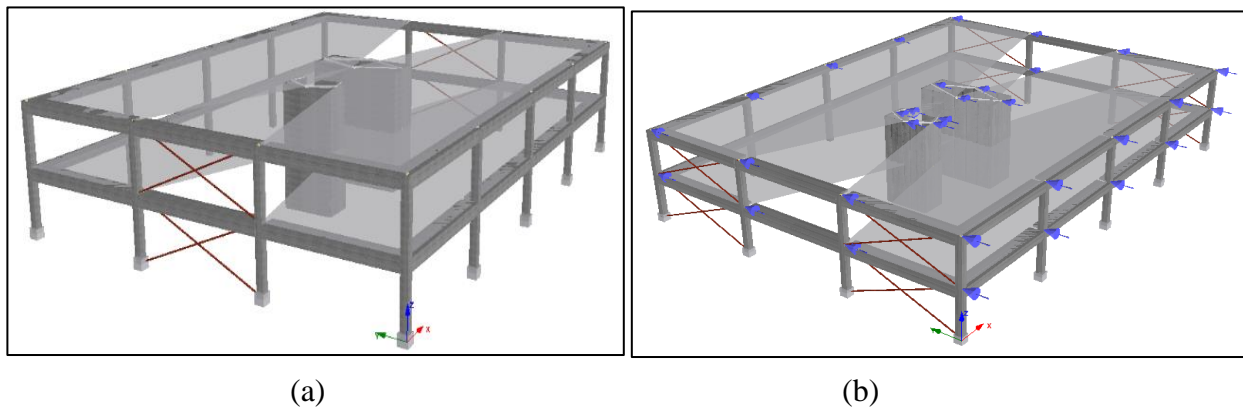


Figure 13: Strengthened with (a) Bracing Layout 1 (b) Bracing Layout 2 with blue arrows representing pushover loads on the structure

5.2. Walls

The effect of addition of shear walls as retrofitting measure was also investigated. Walls were added in the strong axis of the building to increase the shear strength of the building. The properties of the added walls are shown in Table 5. The building model with the addition of shear walls is shown in Figure 14.

Table 5 : Design and Material Properties of new wall

New Wall								
Width (mm)	Thickness (mm)	Cover (mm)	f'_c (MPa)	f_y (MPa)	$d_{\text{longitudinal}}$ (mm)	$S_{\text{longitudinal}}$ (mm)	$d_{\text{transverse}}$ (mm)	$S_{\text{transverse}}$ (mm)
8400	400	25	50	500	16	230	10	100

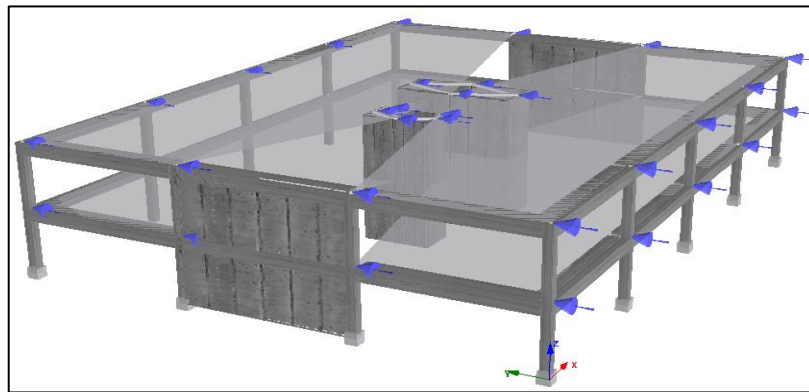


Figure 14: Proposed wall placement for wall retrofit Scheme

5.3. X-Bracings and Wall Combination

It is known that adding bracings and walls as retrofit options individually can help increase the stress and stiffness of a structure. Thus, by combining them together, there is a possibility for even more stability and strength increase.

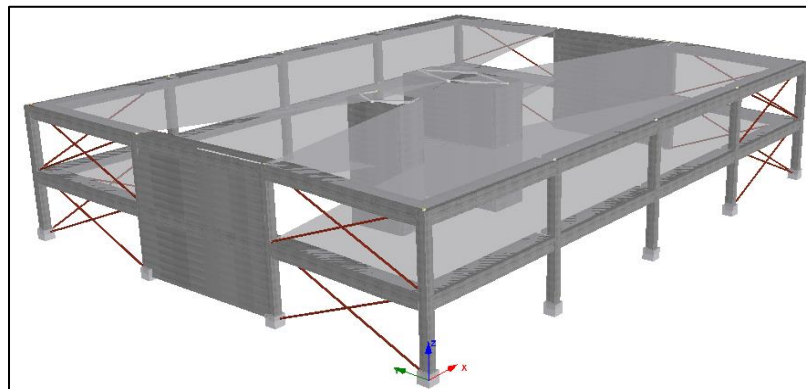


Figure 15: Proposed wall+ Bracings Combination

6. PERFORMANCE LEVELS

Seismic assessment of an archetypal reinforced concrete building was evaluated by studying its response with respect to specified performance levels. Performance levels define the extent of damage considered to be acceptable for different limit states. In this study, four performance levels have been defined (Table 6), and these have been used to conduct the assessment of limited ductile RC buildings. The choice of these levels has been adapted from the recommendations in Amirsardari (2018) and Menegon et al. (2019).

Table 6: Summary of Performance levels selected (Menegon, 2019 & Amirsardari, 2018)

Performance Level	Damage State	Description	Force-Displacement Behaviour	Concrete Strain	Steel Strain	Inter-Storey Drift (%)
Immediate Occupancy	Slight	<ul style="list-style-type: none"> Minimal Damage. Hairline cracks. Concrete and steel strains still within elastic zone. 	Point of First Yield	0.0015	0.005	0.2
Damage Control	Moderate (Repairable)	<ul style="list-style-type: none"> Critical load resisting elements reaching yield Concrete reaches maximum strength Minimal reinforcement inelastic strains 	Effective Yield	0.002	0.01	0.5
Life Safety	Extensive (Severe)	<ul style="list-style-type: none"> Large cracks and spalled concrete Partial collapse of some elements Significant inelastic behaviour in concrete and reinforcement 	Lateral Load Failure (20% reduction from peak strength)	0.006	0.05	1
Collapse Prevention (confined)	Complete (Partial Collapse)	<ul style="list-style-type: none"> Permanent lateral deformation/brittle failures Loss of stability Imminent danger of collapse 	Ultimate drift (50% reduction from peak strength)	0.008	0.1	2

In this paper, some results are compared against results from Amirsardari (2018) for verification purposes, thus, the same basis for determining the performance level of the buildings has been used. That means that when the first component of the building reaches a specific structural damage limit or when the inter-storey drift limit has been exceeded, the structure was considered

to be at that performance level. The same applies for the collapse performance limit. Collapse of the building was determined based on the first component within the building to reach the limit states. However, the actual collapse of the building might be further away from the point identified, depending on whether or not the failed elements are critical.

It is important to note that SeismoStruct does not model column shear degradation (which starts to occur at the life safety performance level) unless a code-based capacity check is applied in the software. Thus, the required residual strength was specified to the corresponding limit state in the code checks.

7. RESULTS

7.1. Unstrengthened Model

The unretrofitted building model was analysed under pushover analysis, using a triangular load pattern, and pushed until failure. The force-displacement response of the building model was presented in the acceleration vs displacement format in Figure 16a and superimposed with the design response spectrum in accordance with AS1170.4-2007 (Standards Australia, 2007) for site class D. The value of design seismic hazard was calibrated such that the demand curve intersects with the force-displacement curve at a certain performance level (as shown in Figure 16a) for collapse performance level. The calibrated Z value was considered the level of intensity measure at which a performance level has been exceeded. The Z value was converted into the maximum response spectral velocity RSV_{max} by the following equations:

$$RSA_{max} = 3.68Z \quad \text{Equation 5}$$

$$RSV_{max} = RSA_{max} \frac{T_1}{2\pi} \quad \text{Equation 6}$$

Where $T_1 = 0.538s$

It is noted that the damping ratio of 5% that was assumed in the construction of the displacement response spectrum is conservative as the building responding in the inelastic range.

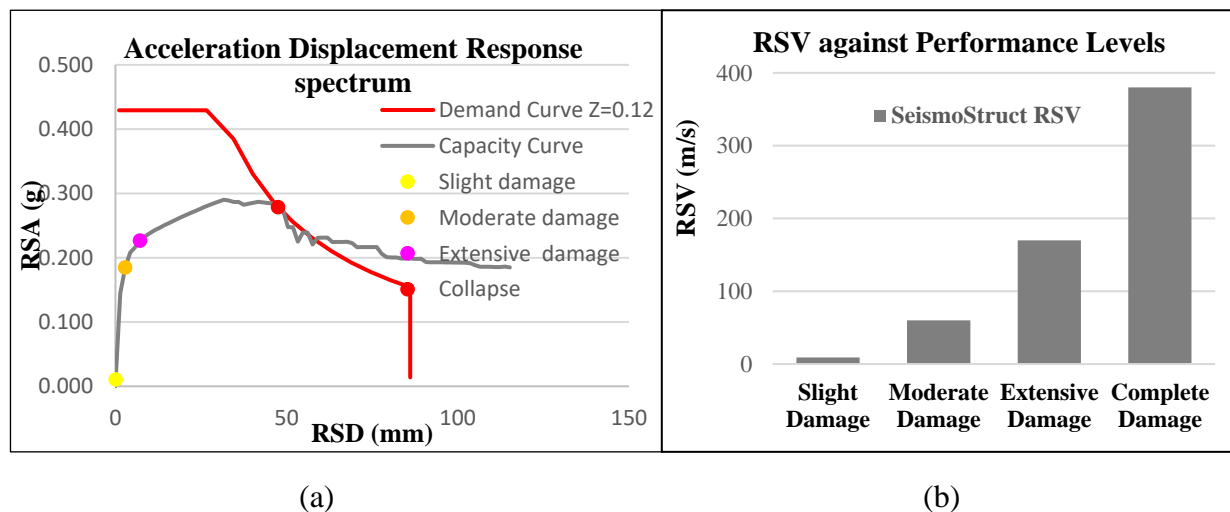
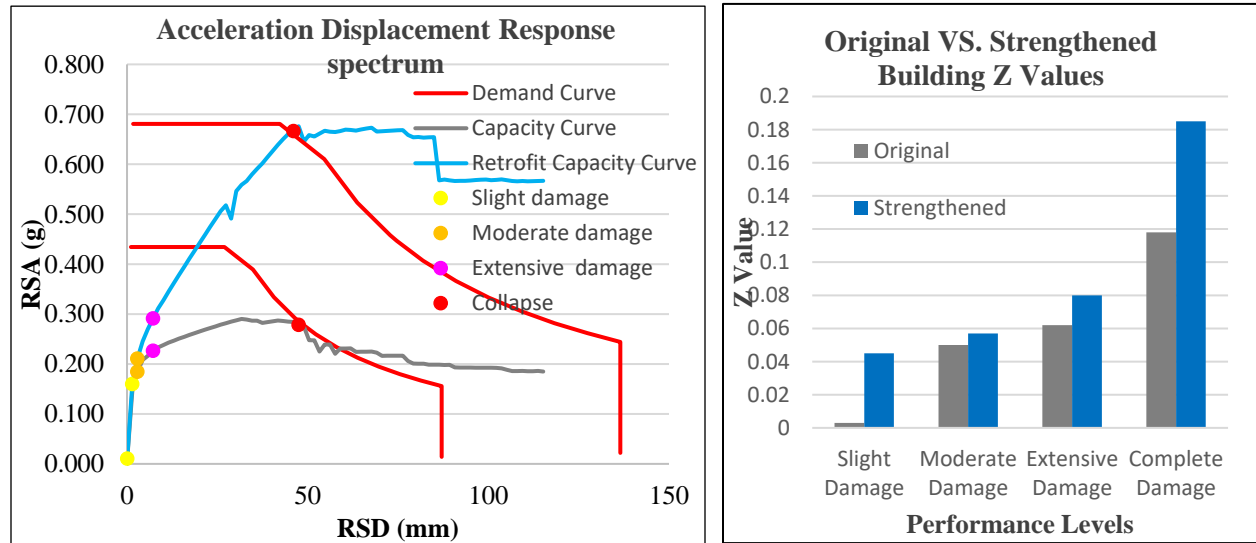


Figure 16: (a) ADRS graph for the unstrengthened model with marked performance points (b) RSV against Performance levels

7.2. The addition of X-Bracings

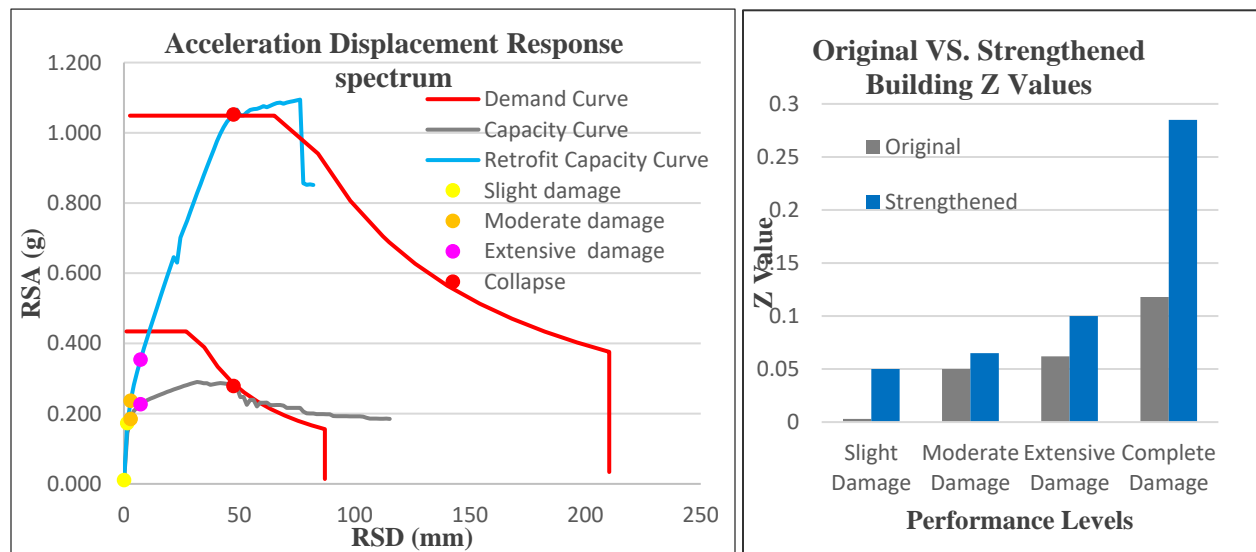
It was observed that bracings, provide consistent increase in the point of slight damage and collapse/complete damage, while there was minimal increase in the other performance points from figures 17 and 18. Moreover, layout 2 provides the higher Z value for the collapse performance point than layout 1, as seen from figures 17-18 (b). This retrofit method might be desirable when the decision-makers' main purpose of the retrofit is to prevent to collapse of the building or to ensure that collapse occurs at a higher seismic hazard value, as well as the increase of torsional stiffness (which effect will be more significant in non symmetric plan buildings).



(a)

(b)

Figure 17: Strengthened with Bracing Layout 1 VS. Original building (a) ADRS (b) Z values



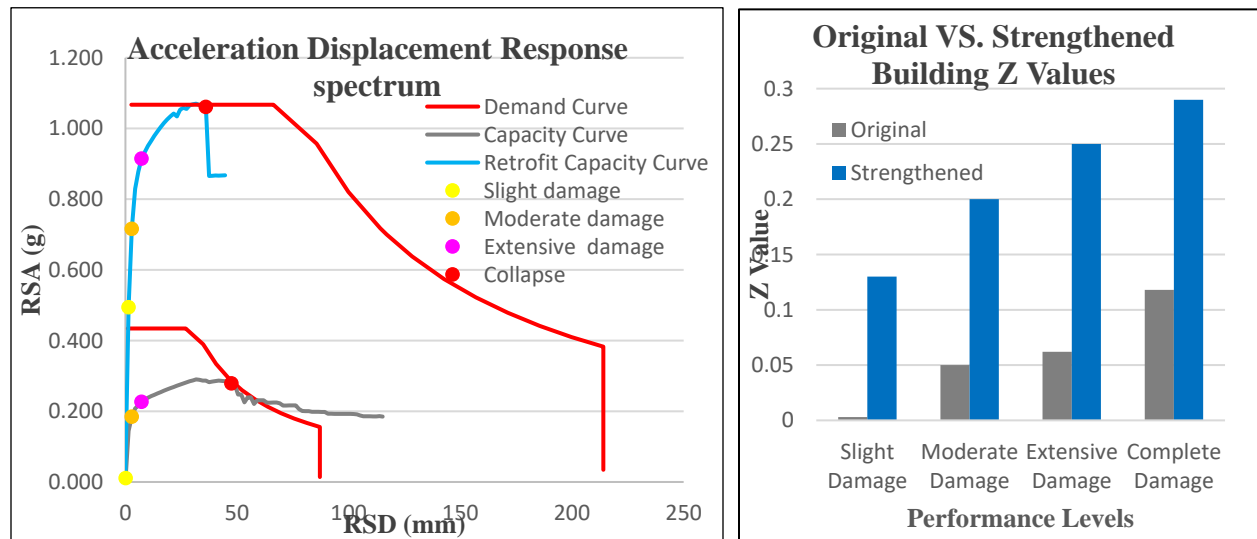
(a)

(b)

Figure 18: Strengthened with Bracing Layout 2 VS. Original building (a) ADRS (b) Z values

7.3. Wall (Strong Axis)

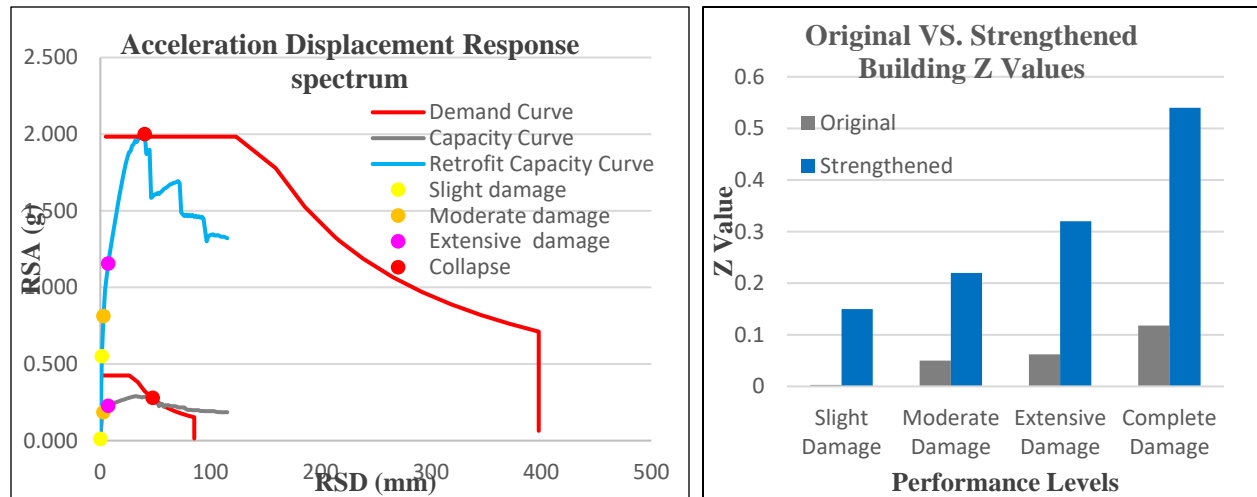
The provision of walls in the strong axis of the building was shown from Figure 19 to have provided a great increase in the performance points, across all the levels. However, it is important to note that the placing of the walls in the strong axis is critical. The walls that are placed in the weak axis would result in no increase in the performance points and aid in the collapse of the building. Hence, the placement of the wall was very critical in increasing the performance points. Note that the Z value of the complete damage performance point was similar to that obtained from Bracings layout 2 (Fig. 18b). hence, this is recommended when better behaviour across all performance levels is required, however, brittle collapse of the structure must be acceptable as the addition of the walls seem to reduce the ductility of the structure.



(a) (b)
Figure 19: Strengthened with Wall VS. Original building (a) ADRS (b) Z values

7.4. X-Bracings + Wall

The combination of walls and bracings together was shown in Figure 20 to provide a higher Z value than any of the individual retrofit options, as expected. There was a consistent increase in the performance points across the first three performance levels, whereas there was a larger increase in the collapse performance point, which was due to the braces contribution (Fig 20b). Compared to the braces and walls retrofit independently, this provided a collapse Z of double the value. If a large Z value is desired, then this combination is effective. Overall, this retrofit ensures better behaviour across all performance levels at much higher seismic hazard levels, which is desirable for buildings of high importance. The addition of bracings also increase the torsional stiffness and reduce the brittle failure that was seen from the addition of walls only.



(a) (b)
 Figure 20: Strengthened with Bracing and Wall Combination VS. Original building (a) ADRS
 (b) Z values

With all the above retrofit methods, the displacement at the performance points remains the same, while the force required to reach that displacement increases, providing greater capacity. Recommendations on the most effective retrofit option can be made, however, more research is required to obtain a more conclusive cost-benefit analysis.

8. CONCLUSION

A two-storey archetypal limited ductile RC building was analysed under pushover analyses using SeismoStruct. The model was validated by comparing results from the analyses against published experimental results. The archetypal RC building was analysed and results from the three-dimensional building model was validated by comparison with results from previous studies. Several retrofitting options were investigated following the same methods. These options included bracings, walls and a combination of walls and bracings. Bracings, with several layouts explored, produced a large improvement in the collapse performance points while the other performance levels remain relatively unaffected. Walls provided better results with increased performance points at all levels. Combination of walls and bracings provide the best results in terms of performance point increase as there was an increase across all levels and a higher increase in the collapse performance points. Further studies are required involving cost-benefit analyses on all of the retrofitting options.

9. REFERENCES

- Almeida, J. P., Tarquini, D., & Beyer, K. (2016). Modelling Approaches for Inelastic Behaviour of RC Walls: Multi-level Assessment and Dependability of Results. *Archives of Computational Methods in Engineering*, 23(1), 69-100.
- Altheeb, A. H. (2016). Seismic drift capacity of lightly reinforced concrete shear walls. (PhD Thesis), Department of Infrastructure Engineering, The University of Melbourne.
- Amirsardari, A. (2018), *Seismic assessment of reinforced concrete buildings in Australia including the response of gravity frames*, PhD Thesis, Department of Infrastructure Engineering, The University of Melbourne.
- Amirsardari, A., Goldsworthy, H. M., & Lumantarna, E. (2017). Seismic site response analysis leading to revised design response spectra for Australia. *Journal of Earthquake Engineering*, 21(6), 861-890. doi: 10.1080/13632469.2016.1210058
- Belejo, A., Bento, R., & Bhatt, C. (2012). Comparison of different computer programs to predict the seismic performance of SPEAR building by means of the SPEAR building Pushover by means Analysis of Pushover Analysis. 15 WCEE Lisboa 2012.
- Beyer, K., Dazio, A., & Priestley, M. J. N. (2008). Inelastic Wide-Column Models for U-Shaped Reinforced Concrete Walls. *Journal of Earthquake Engineering*, 12(sup1), 1- 33. doi: 10.1080/13632460801922571
- Bolea, O. (2016). The Seismic Behaviour of Reinforced Concrete Frame Structures with Infill Masonry in the Bucharest Area. *Energy Procedia*, 85, 60-76.
- Bousias, S. N, Spathis, L.A., Fardis, M.N. "Concrete or FRP Jacketing of Columns with Lap Splices for Seismic Rehabilitation", *Journal of Advanced Concrete Technology*, Vol. 4, No. 3, 2006, pp. 1-14.
- Carvalho, G., Bento, R., & Bhatt, C. (2013). Nonlinear static and dynamic analyses of reinforced concrete buildings—comparison of different modelling approaches. *earthquake and structures (EAS)*, An International Journal, Techno-Press, Editors-in-Chief: Stavros A. Anagnostopoulos (European ed.), Izuru Takewaki (Asia-Pacific ed.), Jerome P. Lynch (American ed.), 4(5), 451-470.
- Caterino, N. & Cosenza, E. (2018). A multi-criteria approach for selecting the seismic retrofit intervention for an existing structure accounting for expected losses and tax incentives in Italy. *Engineering Structures*. 174. 840-850. 10.1016/j.engstruct.2018.07.090.
- Dias-Oliveira, J., Rodrigues, H., & Varum, H. (2016). Seismic assessment of low ductile RC structures: buildings from before the modern seismic codes. *Engineering Computations*, 33(4), 1282-1307.
- Hoult, R. (2017), *Seismic Assessment of Reinforced Concrete Walls in Australia*, PhD thesis, University of Melbourne, Melbourne.
- Hoult, R., Goldsworthy H.M., and Lumantarna, E. (2015). *Improvements and difficulties associated with seismic assessment of infrastructure in Australia*. Paper presented at the Bushfire and Natural Hazards CRC & AFAC conference, Adelaide, Australia.
- Huang, W. et al. (2008). *A Case Study Of Performance-Based Seismic Evaluation And Retrofit Of An Existing Hospital Building In California, U.S.* Paper presented at the Proceedings of the 14th World Conference on Earthquake Engineering, Beijing, China.
- Hussain, R. , Wasim, M., Hasan, S. (2016) *Computer Aided Seismic and Fire Retrofitting Analysis of Existing High Rise Reinforced Concrete Buildings*, Netherlands: Springer.

- Lu, Y., Henry, R. S., Gultom, R., & Ma, Q. T. (2016). Cyclic testing of reinforced concrete walls with distributed minimum vertical reinforcement. *Journal of Structural Engineering*, 1-17.
- Lynn, A., Moehle, J.P., Mahin, S.A., and Holmes, W.T., "Seismic Evaluation of Existing Reinforced Concrete Building Columns", *Earthquake Spectra*, vol. 12, No. 4, Nov. 1996, pp. 715-739
- Menegon, S. J., Tsang, H. H., & Wilson, J. L. (2015). *Overstrength and ductility of limited ductile RC walls: from the design engineers perspective*. Paper presented at the Proceedings of the Tenth Pacific Conference on Earthquake Engineering, Sydney, Australia
- Menegon, S.J. (2018), *Displacement Behaviour of Reinforced Concrete Walls in Regions of Lower Seismicity*, PhD Thesis, Department of Infrastructure Engineering, The University of Melbourne.
- Menegon, S.J., Tsang, H.H., & Lumantarna, E et al. (2019) Framework for seismic vulnerability assessment of reinforced concrete buildings in Australia, *Australian Journal of Structural Engineering*, 20:2, 143-158.
- Priestley, M. J. N., Calvi, G. M., & Kowalsky, M. J. (2007). *Displacement-based seismic design of structures*. Pavia, Italy, IUSS Press.
- Raza S., Menegon S.J., Tsang H.H., Wilson J.L. (2018) Experimental Testing Program to Investigate the Collapse Drift Capacity of Limited Ductile High-Strength RC Columns. In: Wang C., Ho J., Kitipornchai S. (eds) *ACMSM25. Lecture Notes in Civil Engineering*, vol 37. Springer, Singapore
- Saatcioglu M. (2006) Seismic Risk Mitigation Through Retrofitting Nonductile Concrete Frame Systems. In: Wasti S.T., Ozcebe G. (eds) *Advances in Earthquake Engineering for Urban Risk Reduction. Nato Science Series: IV: Earth and Environmental Sciences*, vol 66. Springer, Dordrecht
- Saatcioglu, M., and Ozcebe, G., "Response of Reinforced Concrete Columns to Simulated Seismic Loading", *ACI Structural Journal*, Jan. – Feb., 1989, pp. 3-12.
- SeismoSoft. (2018a). *SeismoStruct 2018 - A computer program for static and dynamic nonlinear analysis of framed structures*. Retrieved from www.seismosoft.com
- SeismoSoft. (2018b). *SeismoStruct User Manual 2018 (Release: 3 Build: 1)*. Retrieved from https://seismosoft.com/wp-content/uploads/prods/lib/SeismoStruct-2018-User-Manual_ENG.pdf
- SeismoSoft. (2018c). *SeismoStruct Verification Report 2018 (Release: 3 Build: 1)*. Retrieved from <https://seismosoft.com/wp-content/uploads/prods/lib/SeismoStruct-2018-Verification-Report.pdf>
- Standards Australia. (2007). *AS 1170.4-2007: Structural design actions, Part 4: Earthquake actions in Australia*. Sydney, NSW: SAI Global
- Sunil, S., Sujith P.S. (2017) 'Seismic Study of Multistorey RC Building With Different Bracings', *International Journal of Innovative Research in Science, Engineering and Technology*, 6(5), pp. 8667-867.
- Takemura, H., and Kawashima, K., "Effect of loading hysteresis on ductility capacity of reinforced concrete bridge piers", *Journal of Structural Engineering* 43 1997, pp. 849-858.
- Tankut T., et al. (2006) In Service Seismic Strengthening Of Rc Framed Buildings. In: Wasti S.T., Ozcebe G. (eds) *Advances in Earthquake Engineering for Urban Risk Reduction. Nato Science Series: IV: Earth and Environmental Sciences*, vol 66. Springer, Dordrecht
- Xiao, Y., and Martirosyan, A., "Seismic Performance of High-Strength Concrete Columns", *Journal of Structural Engineering*, March, 1998, pp. 241-251.