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COST-EFFECTIVE MITIGATION STRATEGY DEVELOPMENT FOR BUILDING RELATED EARTHQUAKE RISK

Annual project report 2017-2018

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Cover: Dust clouds of the Feb 2011 Christchurch earthquake (© Gillian Needham)

TABLE OF CONTENTS

EXECUTIVE SUMMARY

This annual report contains a summary of 12-month research undertaken by 4 partner institutions towards the development of cost-effective seismic retrofit methods for vulnerable Australian buildings.

Progress has been made in 3 complementary fronts to:

1) Understand the existing unreinforced masonry (URM) and limited ductile reinforced concrete (LDRC) building vulnerabilities and methods to address them through seismic retrofit;
2) Risk assessment of the building stock through development of an economic loss model; and
3) Advance an end user focused research utilization project in the area of community risk reduction. This is done through an Earthquake Mitigation Case Study for the historic town of York in Western Australia.

The first of the above components is being researched in the Universities of Adelaide, Melbourne, and Swinburne. This work includes investigation of existing building seismic capacities and development of retrofit techniques. The second area is being studied by Geoscience and the work includes estimating direct and indirect losses associated with building damage and benefits from seismic retrofit. The last component is completed utilizing the research findings in the two other areas.

Finally, using the new damage loss models and costings for seismically retrofitting buildings, recommendations are made for the development of seismic retrofit guidelines and policy based on the strong evidence base developed.

END USER STATEMENT

Leesa Carson, *Geoscience Australia, Commonwealth*

During the past 12 months significant progress has been made towards one of the proposed End User projects. As detailed under "Conference and workshop attendance", researchers from GA and Adelaide have been working towards the WA-based End User program. Some of the engagement activities have included a travel to York town and:

- Introduction of Department of Fire and Emergency Services (DFES) personnel to foot survey techniques
- Interview with local newspaper with ensuing article
- Distribution of project flyer through the Council and the York Society to the public
- Proposed briefing at Australian Earthquake Engineering Society (AEES) conference tour later in the year
- Preparation and submission of a joint abstract for the upcoming AFAC conference, with the End User delegates being speakers

INTRODUCTION

This project arose out of the on-going research efforts by the group involving structural engineering academics at the Universities of Adelaide, Melbourne and Swinburne with Geoscience Australia experts all working towards seismic risk reduction in Australia. Most of the research team are actively involved in the revision to the Australian Earthquake Loads standard (AS1170.4) as well as being members of the Australian Earthquake Engineering Society which is a Technical Society of Engineers Australia. The devastating impact of the 2010 – 11 earthquakes in the Christchurch region on the New Zealand economy and society has further motivated this group to contribute to this CRC's aims of risk reduction for all natural hazards in Australia.

This project addresses the need for an evidence base to inform decision making on the mitigation of the risk posed by the most vulnerable Australian buildings subject to earthquakes. While the focus of this project is on buildings, many of the project outputs will also be relevant for other Australian infrastructure such as bridges, roads and ports, while at the same time complementing other 'Natural Hazards' CRC project proposals for severe wind and flood.

Earthquake hazard has only been recognized in the design of Australian buildings since 1995. This failure has resulted in the presence of many buildings that represent a high risk to property, life and economic activity. These buildings also contribute to most of the post-disaster emergency management logistics and community recovery needs following major earthquakes. This vulnerability was in evidence in the Newcastle Earthquake of 1989, the Kalgoorlie Earthquake of 2010 and with similar building types in the Christchurch earthquake. With an overall building replacement rate of 2% nationally the legacy of vulnerable building persists in all cities and predominates in most business districts of lower growth regional centers.

The two most vulnerable building types that contribute disproportionately to community risk are unreinforced masonry and low ductility reinforced concrete frames. The damage to these will not only lead to direct repair costs but also to injuries and disruption to economic activity.

This research project will draw upon and extend existing research and capability within both academia and government to develop information that will inform policy, business and private individuals on their decisions concerning reducing vulnerability. It will also draw upon New Zealand initiatives that make use of local planning as an instrument for effecting mitigation.

WHAT THE PROJECT HAS BEEN UP TO

CONFERENCE AND WORKSHOP ATTENDANCE

BNH CRC Showcase (04 Jul 17) – Mark Edwards presented an overview of the WA-based End User project and the presentation was followed by a brief description from Paul Martin, CEO of York Shire Council. The presenters then met with University of Adelaide researchers through a laboratory tour followed by a project meeting.

Project workshop 1 (30 Nov 17) – Mark Edwards, Martin Wehner, and Mohanty Itismita from Geoscience and a number of researchers from University of Adelaide met in Adelaide in planning for WA End User project works and engagement programs. Others, who participated via teleconference, were Paul Martin, CEO of York Shire Council, Stephen Gray (WA DFES), and the media team from Geoscience.

Project workshop 2 (Dec 17) – Martin Wehner and Mike Griffith travelled to York in late 2017 planning for WA End User project surveys.

Project activities/workshop 3 (early 2018) - Researchers from Geoscience and Adelaide travelled to York, WA to meet with several stakeholders including Shire of York, York Society, York Business Association, DFES, and public (through public outreach sessions). Project works, including foot survey of buildings and interrogation of council register of heritage buildings, were completed and preparations were made for next stages of collaboration including a joint paper proposal for AFAC 2018.

AFAC'17 (04-06 Sep 17) – Hossein Derakhshan, Alireza Mehdipanah, and Mark Edwards attended with 3 posters.

AEES17 (24-26 Nov 17) – 5 papers including a Keynote were presented at the Australian Earthquake Engineering Society (AEES) conference, which was held in late November in Geoscience Australia (Canberra). The presentations included a keynote paper by Mike Griffith on performance expectation from Australian unreinforced masonry buildings.

10AMC (11-14 Feb 18) – One paper was presented on the acceleration response of unreinforced masonry buildings in the 10^{th} Australasian Masonry Conference, which was held in Sydney.

NUMERICAL INVESTIGATION OF ACCELERATIONS APPLIED ON UNNREINFORED MASONRY COMPONENTS

A research was undertaken to estimate peak floor acceleration (PFA) for low-rise URM buildings with flexible diaphragms. These accelerations are applicable to parts and components that are subject to earthquake movements indirectly through building shaking. It is important to reliably assess PFA as elevated URM components pose a significant falling hazard in earthquake. Four building typologies (Figure 1) were created and analyzed through a parametric study.

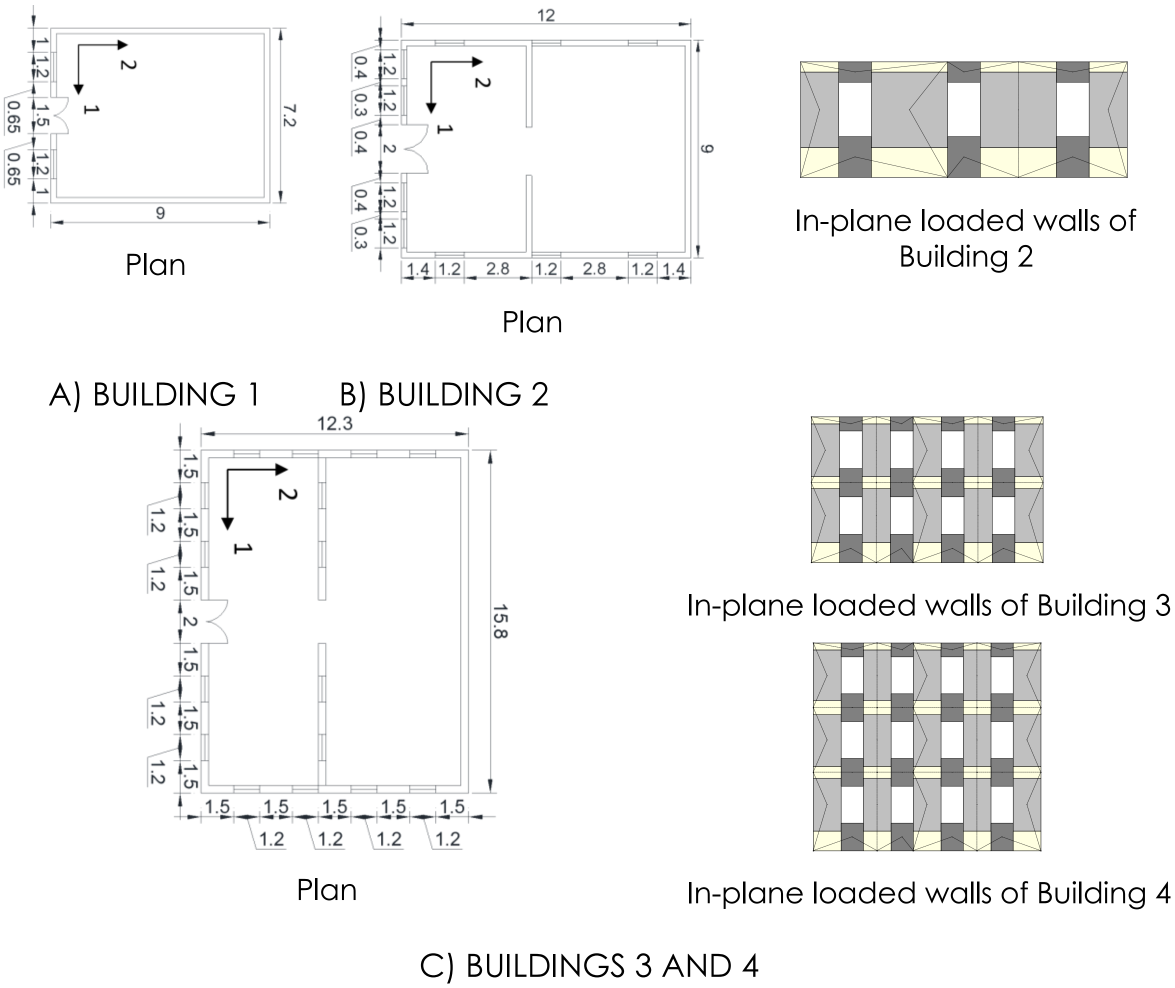


FIGURE 1: CASE STUDY BUILDINGS

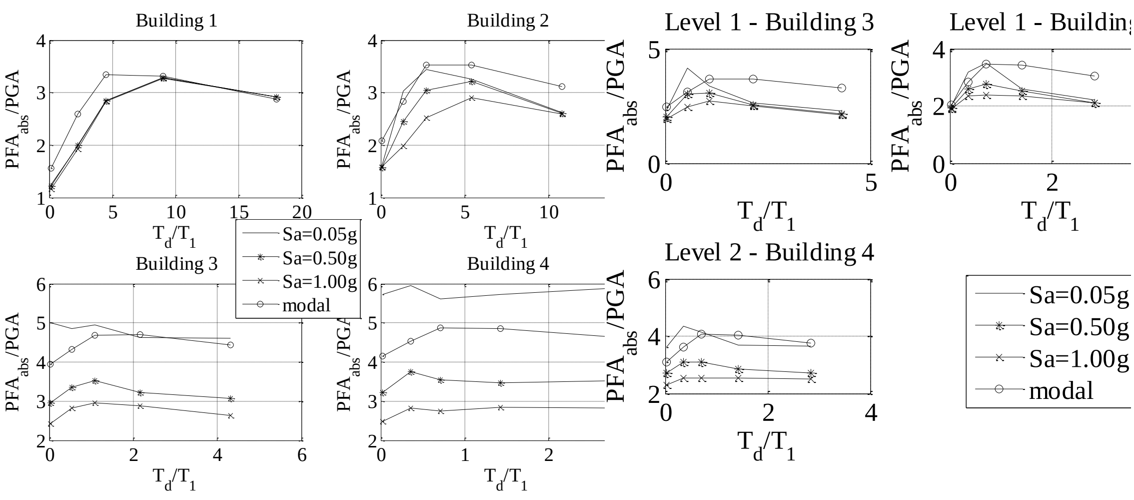
Several levels of diaphragm in-plane stiffness were assumed in the modelling. Default material properties for existing timber floors are set out in (ASCE 2014) in the form of a characteristic shear stiffness, Gd, with a minimum value of 350 kN/m. However, further in situ testing of URM buildings in New Zealand (Giongo et al. 2014) has suggested values up to a third of this stiffness depending on the condition (e.g. decay in timber joists) of the diaphragm. A lower bound of Gd=150 kN/m (D1) was used in this research. As detailed in Table 1, four other cases of diaphragm stiffness were also studied, including a strengthened timber floor (D4) and the previously discussed case of rigid diaphragms (represented as D5 in Table).

Analysis results (Figure 2) shows that for almost all the analysis cases, the PFA to peak-ground-acceleration (PGA) ratio increases to reach a peak value and then reduces as the diaphragm becomes increasingly flexible. This effect is more pronounced for the single-storey buildings. For Building 1, the amplification factor increased by 175% from 1.2 for diaphragm case D5 (rigid) to a value of 3.3 for the diaphragm case D2. The PFA in lower floors of the multi-storey buildings is also affected by diaphragm vibrations as the wall-related vibrations are insignificant due to the first mode shape. Figure 2 also shows the results from a predictive model that can predict the peak floor acceleration with about 30% error but that is still under study.

Table1: Range of diaphragm stiffnesses

Designation	Description	Assumed Gd, kN/m	Ref. period Td*, sec
D1	As-built with single straight sheathing	150	1.08
D2	As-built with single diagonal sheathing; unchorded	600	0.54
D3	As-built with double straight sheathing; chorded	2400	0.27
D4	Single straight sheathing strengthened with 19 mm plywood overlay with substantial edge nailing	9600	0.13
D5	Large stiffness representing a rigid diaphragm	3 x 106	0.01

* Calculated using the diaphragm stiffness and the combined mass of the diaphragm and the tributary mass of the out-of-plane loaded walls (little variations for different buildings ignored)



a) Roof   b) Floors

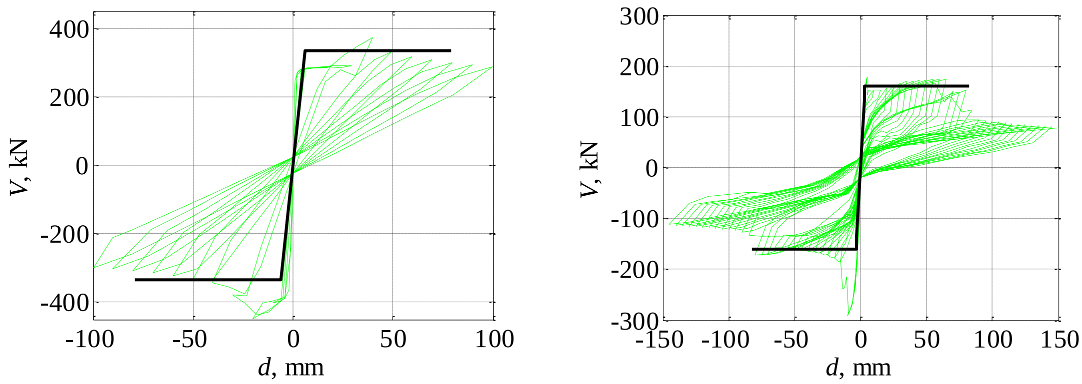
Figure 2: Peak floor accelerations for buildings with flexible diaphragms

In conclusions, it was found that the peak floor accelerations in buildings with flexible diaphragms can be up to nearly 2 times greater than that in a building with rigid floors. Therefore, it is clear that the code approaches (e.g. Australian seismic loading code, AS1170; AS 2007) that have been developed for ignoring floor vibrations cannot be applied to buildings that include flexible diaphragms.

STUDY OF DRIFT-DAMAGE RELATIONSHIPS FOR URM BUILDINGS

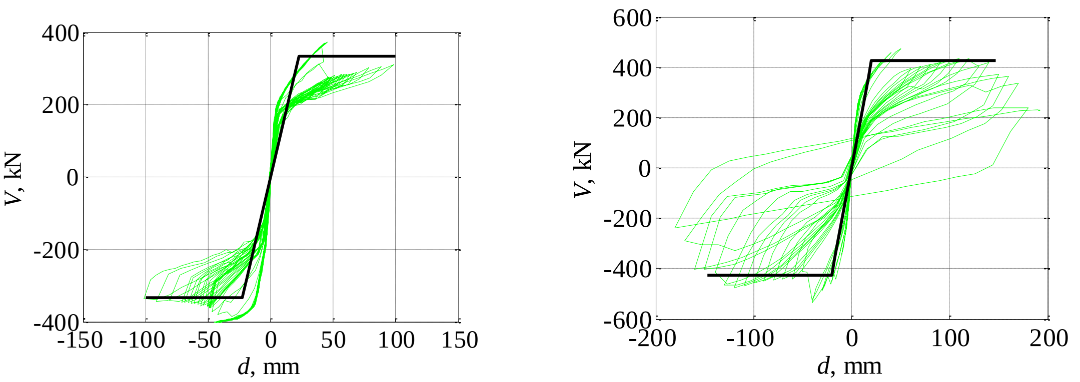
The same buildings as shown in Figure 1 were studied to estimate a relationship between URM building damage and imposed lateral displacements. The building damage level was represented in 5 increments from D1 (Immediate Occupancy, IO) to D5 (Collapse). The obtained relationships were compared and contrasted with the values found in masonry literature including in a guidelines published by American Society for Civil Engineering (ASCE 2014).

Pushover curves (Figures 3 and 4) were obtained and the condition of the building damage as reflected in numerical model "damage parameters" were observed. As part of the utilized numerical technique, masonry walls are modeled as piers or spandrels and these damage parameters are part of the material model. Gradual increments in these parameter are an indicative of the state of shear damage.



a) Building 1 b) Building 2

Figure 3: Pushover curves for Buildings 1 and 2



a) Building 3 b) Building 4

Figure 4: Pushover curves for Buildings 3 and 4

Results are presented in two forms of damage vs. building drift (Table 2) and damage vs critical storey drift (Table 3).

A direct comparison of the results for 'critical' storeys in the 4 studied buildings with ASCE recommendations suggests that the latter corresponds to a conservative evaluation of storey drift ratios responsible for different damage states.

The average storey drift ratio that were obtained for damage level D1 are consistent with the values recommended in ASCE (2014), e.g. 0.23% for D1 vs. 0.30% from Figure 2 for IO limit state.

Larger storey drift ratios were obtained for Collapse Prevention (D4), 2.25% vs. 1% recommended in ASCE. The drift ratio for intermediate D2 and D3 levels obtained in this study (0.64% and 1.34%) also exceed ASCE recommendations (0.6% for D3).

Table 2: Summary of building drift ratios (%) vs damage levels

Building model	1	2	3	4	Average (COV)
D1	0.14	0.07	0.26	0.18	0.16
D2	0.71	0.35	0.59	0.44	0.52
D3	1.29	1.18	0.97	0.72	1.04
D4	1.86	1.93	1.29	1.31	1.60

Table 3: Summary of storey drift ratios (%) vs damage levels

Building model	1	2	3		4			Average for critical storey
Building Level	Lvl. 1	Lvl. 1	Lvl. 1	Lvl. 2	Lvl. 1	Lvl. 2	Lvl. 3	---
D1	0.14	0.07	0.5	0.1	0.3	0.2	0.1	0.23
D2	0.71	0.35	1.0	0.2	0.7	0.5	0.2	0.64
D3	1.29	1.18	2.0	0.1	0.8	0.9	0.5	1.34
D4	1.86	1.93	2.7	0.1	1.1	2.5	0.6	2.25

One critical aspect that needs to be addressed in building analysis is the potentially uneven distribution of structural damage with building height, which needs to be addressed before expected drifts on URM walls can be determined. This would not be an issue if the building can indeed be idealised as a SDOF 'regular' structure but the definition of structural irregularity is not very well understood in the context of URM buildings that can have walls with different thicknesses in different stories.

GENERALISED FORCE METHOD FOR DISPLACEMENT DEMAND ESTIMATES ON IRREGULAR BUILDINGS

Generalised force method has been developed to provide estimates of displacement demand of multi-storey buildings with vertical irregularities. The effects of higher modes have been taken into account based on generalised modal values presented in Figure 5 and by assuming the second modal period (T_2) that is 0.25 of the fundamental natural period (T_1) of the building. The values in Figure 5 are the mean values of results obtained from dynamic analyses conducted on multi-storey buildings of varying height. The buildings are supported by reinforced concrete walls and moment resisting frames and feature vertical irregularities caused by discontinuities of the columns. Several building configurations with varying contribution of moment resisting frames to the lateral stiffness of the building were included in the analyses. Results from the studies presented in the form of modal values versus normalised height of the buildings are presented in Figure 6 for the first and second mode of response. Expressions have been introduced to provide estimates of displacement, inter-storey drift and inertia forces taking into account the higher modes effects. Figure 8 presents the displacement profile, inter-storey drifts and storey shears for a 20-storey building (the typical floor is shown in Figure 7). Comparison with results from dynamic analysis of the building demonstrates that the modified GFM is able to reasonably estimate the displacement and shear demands on the 20-storey building.

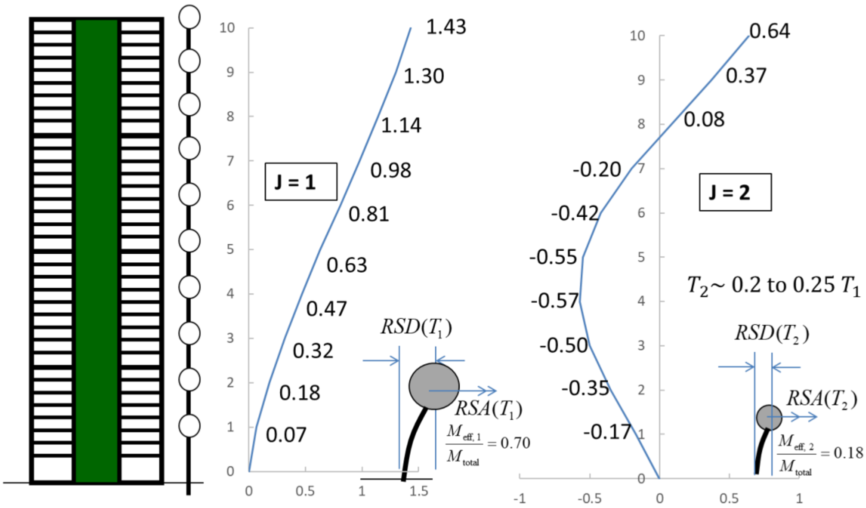
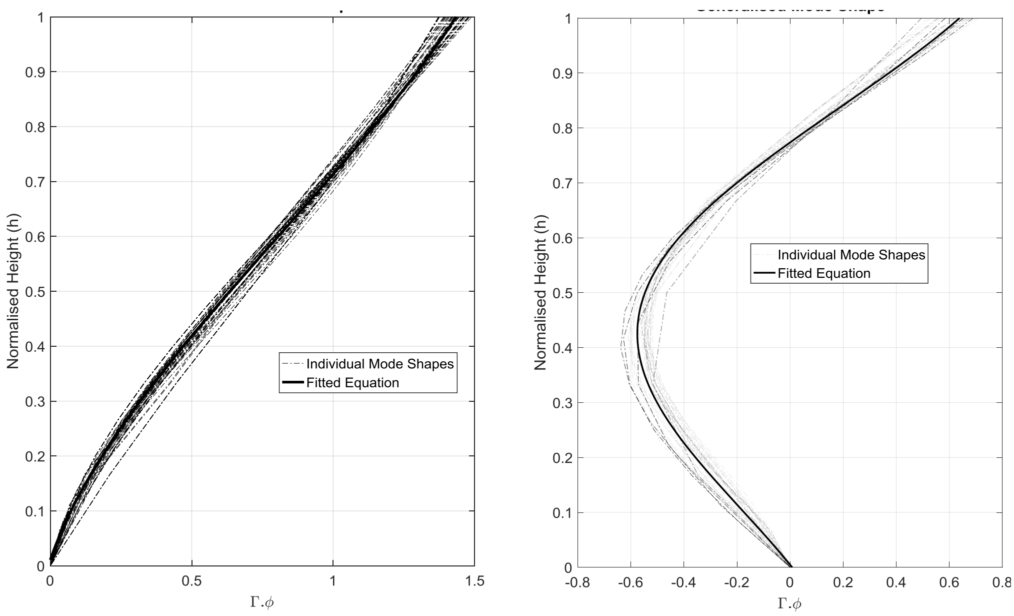


Figure 5: Generalised modal values ($\Gamma_j . \phi_j$)



(a) first mode (b) second mode

Figure 6: Modal displacements obtained from parametric studies

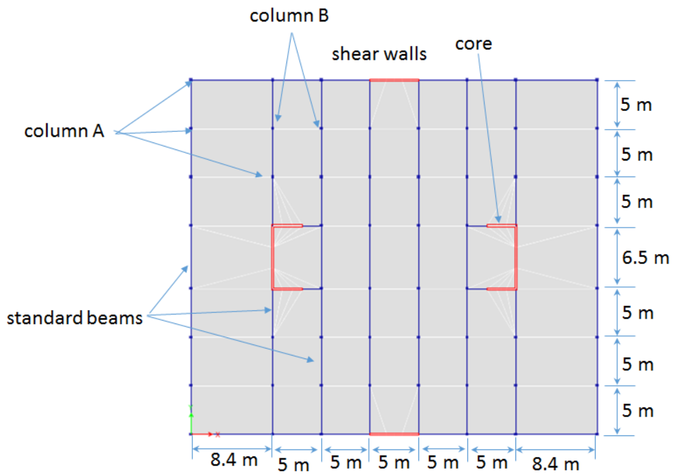
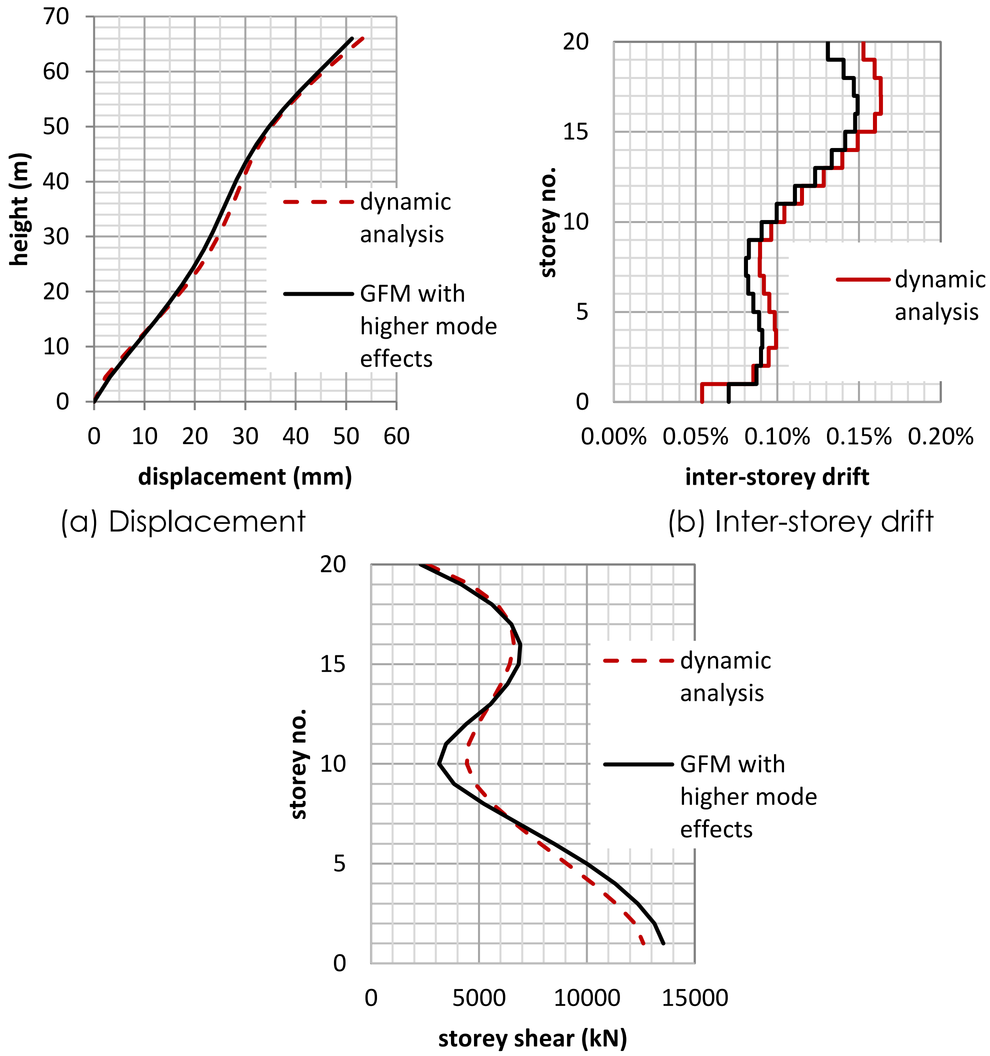


Figure 7: Typical floor plan of the torsionally balanced 20-storey building



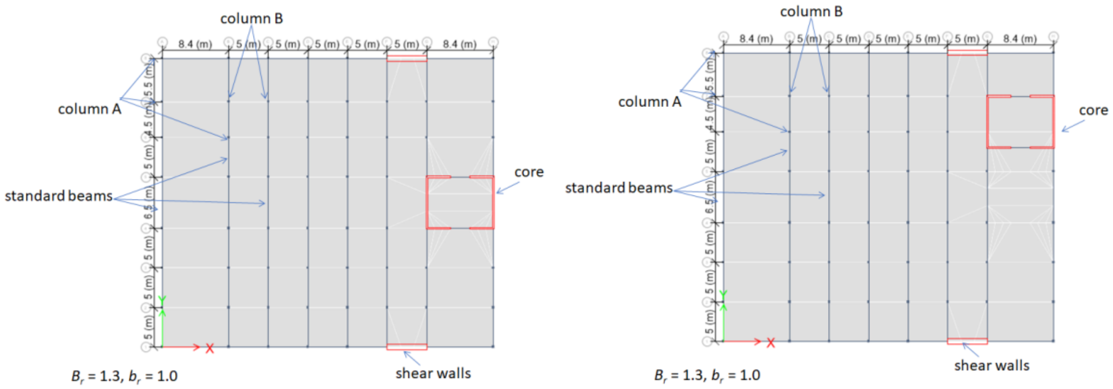
(a) Displacement (b) Inter-storey drift

(c) Storey shear

Figure 8: Results from GFM with higher mode effects, 20-storey building

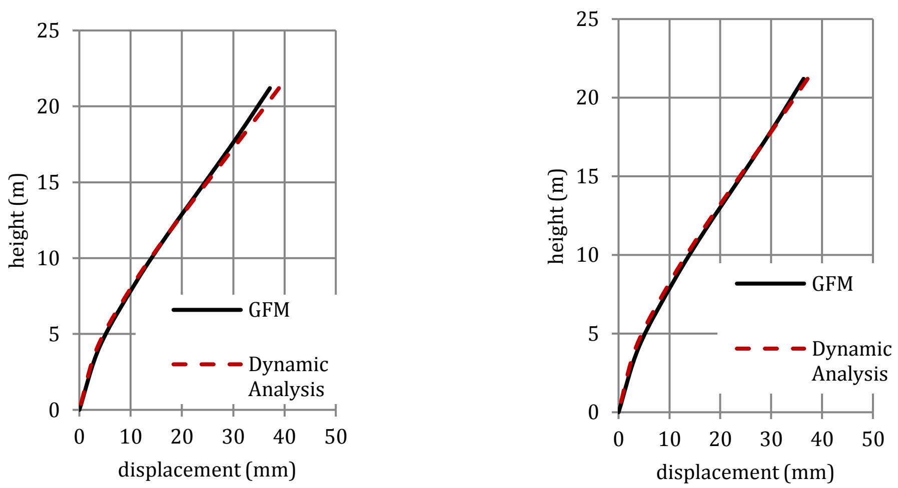
The Generalised Force Method has been extended to provide estimates for multi-storey buildings with plan irregularities. Expressions have been derived to provide estimates of displacement demands at the edges of torsionally unbalanced buildings with uni-axial and bi-axial asymmetry based on the following parameters: i) e_x is the eccentricity perpendicular to the direction of motion; ii) *J* is the torsional mass of inertia of the TU building; iii) Parameter "*b*" (= $= \sqrt{K_\theta / K_y}$), which is used to represent the torsional stiffness properties of the TU building; and iv) a $(= \frac{K_x}{K_y})$.

A method to idealise multi-storey buildings with varying storey eccentricity and stiffness into single storey building model has also been proposed. The method requires determining the value of the eccentricity e_{xr} and that of the torsional stiffness parameter b_r noting that the dynamic torsional response behaviour of the building models are characterised by these two parameters. The expressions based on the values of the torsion parameters can be used to determine the amplification factors which are applied to the floor displacements to provide estimates of the maximum displacement demands of the TU multi-storey buildings. Figure 10 presents comparison between results from GFM and dynamic analyses of uni-axial and bi-axial asymmetric buildings. The typical plan view of the building is presented in Figure 9. The comparison shows that the method is able to approximate the maximum displacement demand of the torsionally unbalanced buildings.



(a) uni-axial asymmetric building (b) bi-axial asymmetric building

Figure 9: Typical floor plan of torsionally balanced buildings



(a) uni-axial asymmetric building (b) bi-axial asymmetric building

Figure 10: Comparison between GFM and results from dynamic analyses

SEISMIC DEMANDS ON PODIUM-TOWER BUILDINGS

Studies have been conducted to investigate the seismic demands of high-rise buildings with a transfer structure (Figure 11). An analytical procedure for predicting the increase in shear demands on the tower walls after the supporting transfer level has been proposed. The shear force increases are the result of the significant strutting forces (F_{STRUT}) developed in the slabs (and beams) connecting adjacent tower walls. Expressions for the strutting force (F_{STRUT}) as illustrated in Figure 12 have been proposed as a function of the differential rotation of the adjacent tower walls about at their bases, the Flexibility Index (FI) and the the differential rotation of the adjacent tower walls about at their bases ($\Delta\theta_{TP}$). The *Flexibility Index* (FI) has been defined as a function of the stiffness of the transfer structure relative to the tower walls stiffness. The differential wall rotation $\Delta\theta_{TP}$ correlates with the angle of drift of the building at mid-height. A 2DOF model of the building tower provided predictions of *Peak Rotational Demand* (PRD) which can be taken as a conservative estimate of $\Delta\theta_{TP}$. The value of F_{STRUT} may then be expressed as the product of *FI,* PRD and $\mathbf{E_C A_{eff}}$ **which is** the axial stiffness of the connecting elements.

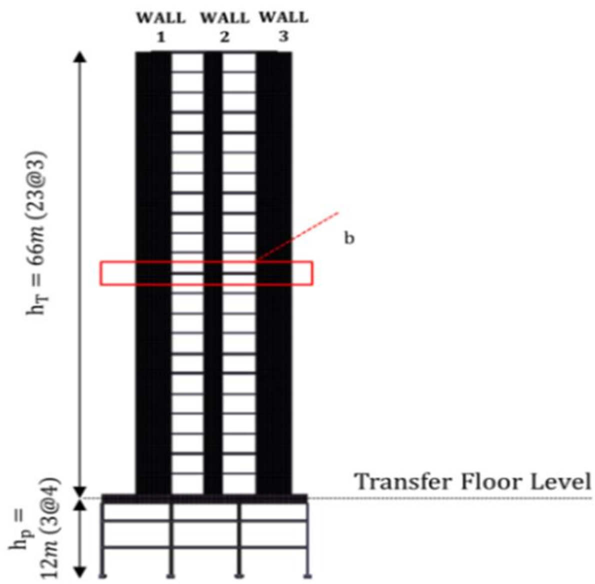


Figure 11: 2D model of the building featuring a transfer plate

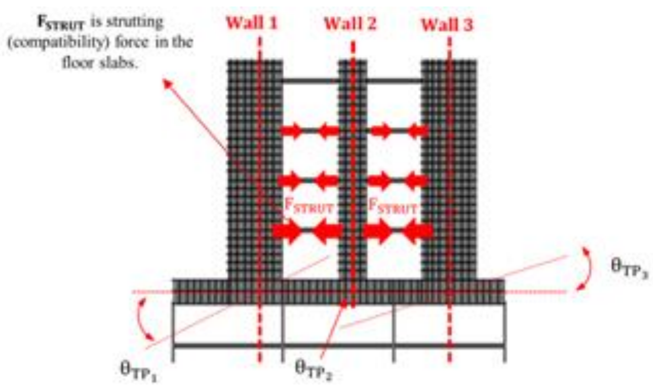
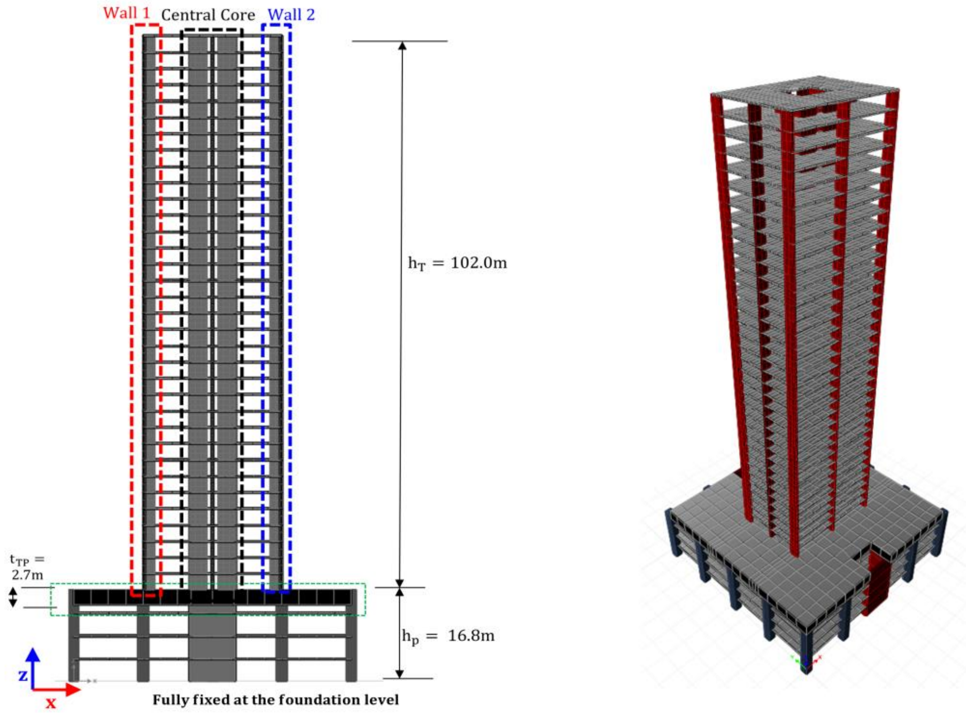


Figure 12: Introducing F_{STRUT}, ε_{STRUT} and $\Delta\theta_{TP}$, where $\Delta\theta_{TP} = \theta_{TP1}$- θ_{TP2}

Three dimensional dynamic analyses were conducted on a building featuring a transfer structure, as shown in Figure 13. Figure 14 presents the shear force distribution on the tower walls 1 and 2 showing an increase in the shear demands at the storey just above transfer force level (indicated by the red line). It is shown that the sharp increase in shear force on the tower walls above TFL was found to be of the order of 500 kN (as shown in Figure 14). The results from dynamic analysis is in good agreement with the prediction of 557 kN made by the proposed simplified method.



(a) Elevation view of the case study building showing the analysed walls

(b) 3D render of the FE model of the case study building

Figure 13: Case study building

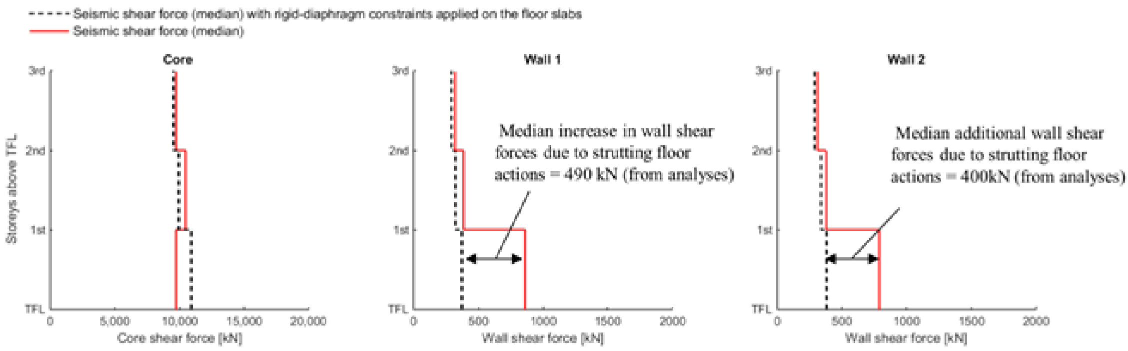


Figure 14: Shear force distribution above TFL from dynamic analyses

NUMERICAL STUDIES TO DEVELOP LATERAL DRIFT MODEL FOR LIMITED DUCTILE REINFORCED CONCRETE COLUMN

Studies have been undertaken to present a detailed lateral load-drift model (pushover curve) that possesses the ability to predict the lateral load-drift behaviour of limited to moderately ductile NSRC as well as HSRC columns.

A detailed lateral load-drift model for RC columns has been proposed, the model is defined by five points, namely, cracking strength, yield strength, ultimate strength, lateral load failure (20% lateral strength degradation) and axial load failure (50% lateral strength degradation) as shown in Figure 15. The model presented includes the expressions for post-peak failure drift that are applicable to both NSRC and HSRC columns. Expressions defining each of point have been developed and calibrated using an extensive database of NSRC and HSRC columns from the literature. These expressions fit the experimental data very well and relate the post-peak drift capacity with the following design parameters: axial load ratio, transverse reinforcement ratio, transverse reinforcement yield strength and concrete compressive strength.

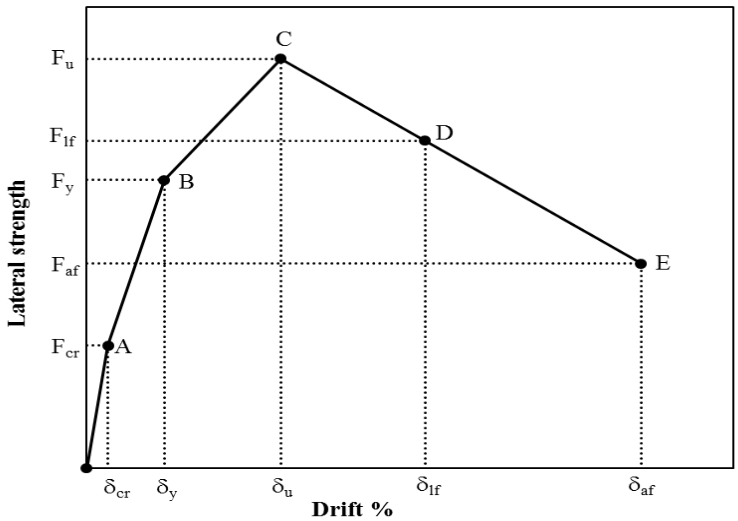


Figure 15: Detailed lateral load-drift model for RC columns

The model is used to plot pushover curve for a cantilever column of 500×500 mm cross-section, having an aspect ratio of 4.0 and reinforced with 8N24 longitudinal bars (ρ_v=1.45%). The 3-legged N10 ligatures with a transverse reinforcement yield strength of f_{yh} =500 MPa are spaced at 250 mm to give a transverse reinforcement ratio by area of ρ_h=0.19%. The variable parameters for this case study are concrete compressive strength: f_c'=25 to f_c'=100 MPa and axial load ratio: n=0.1 to n=0.4. Results are presented in Figure 16 highlighting the significant impact of the axial load ratio on the post-peak drift capacity of the RC column.

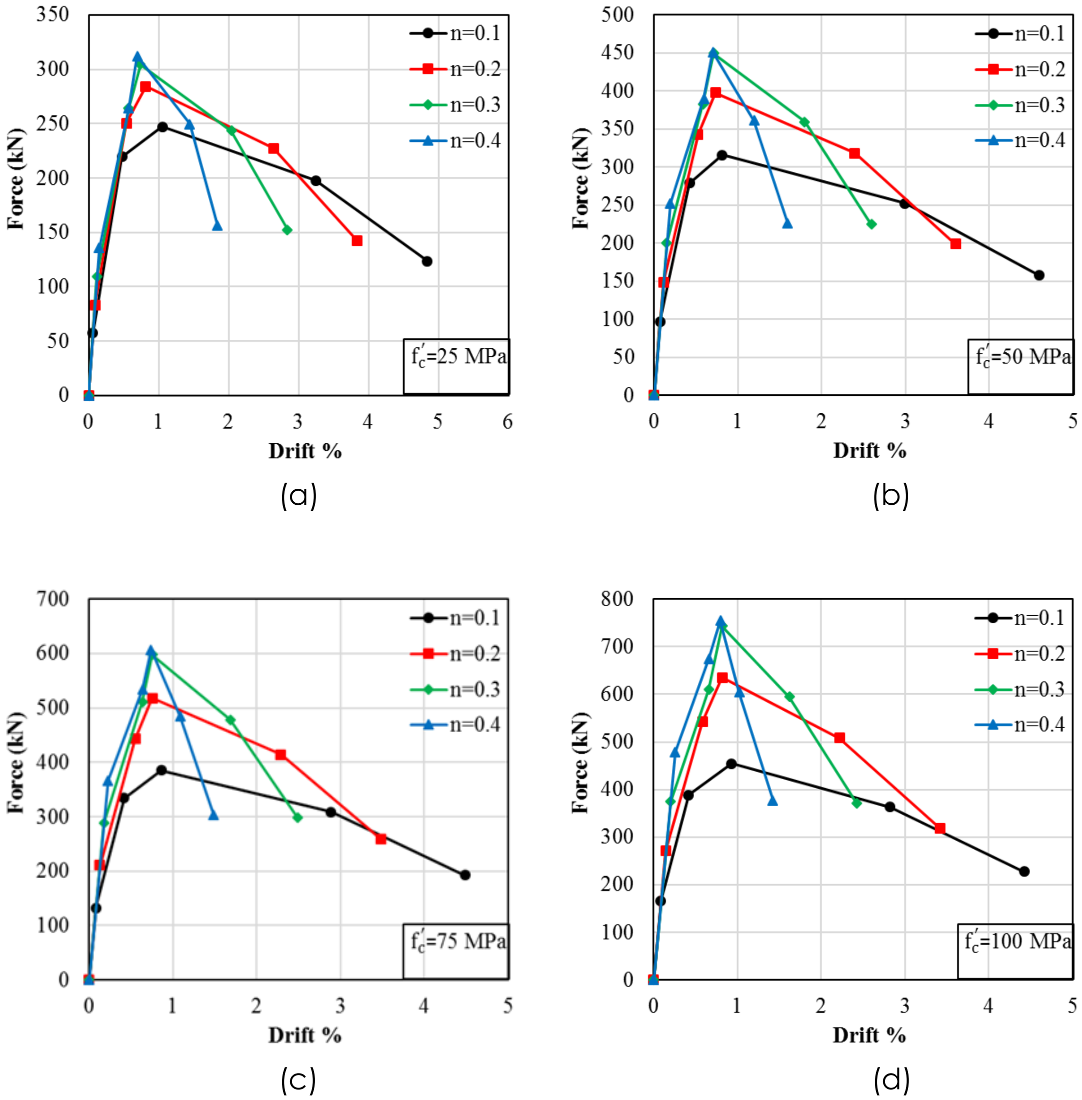


Figure 16: Lateral load-drift curves for case study example

SEISMIC FRAGILITY FUNCTIONS FOR PODIUM-TOWER BUILDINGS

Fragility curves have been constructed on two groups of building models. The first group comprises two buildings that feature a setback in the floor plan above the podium level (buildings designated by SB-1 and SB-2) as shown in Figure 17. The second group includes two building models incorporating a transfer plate at the level of the podium (designated by TS-1 and TS-2) as shown in Figure 18. For all 2D sub-frame models, the tower structure comprises of three walls connected by floor slabs.

The fragility curves were constructed based on incremental dynamic analyses using a suite of 40 ground motion records consisting of artificial and historical records. Scalable intensity measures (*IM*) of RSDmax, PGV and PGA were selected to quantify the intensity of the ground motion demand on the building. The parameter RSD_{max} is the maximum ordinate of the displacement spectrum of the ground motion record. The parameter PGA is the maximum value of the ground acceleration time trace and PGV is the maximum value of the ground velocities. The maximum inter-storey drift (*ISD*) is a common choice for the engineering demand parameter (*EDP)* in tall buildings and has been extensively used to evaluate seismic damages on shear-critical walls and non-structural components in the building. In this study, the *EDP* has been specified as the maximum inter-storey drift ratios (*ISD*) occurring above the transfer floor or the podium interface levels. This choice of the *EDP* with the main focus on the storeys above the level of the interface is founded on the observations reported that most of the critical seismic damages have been reported in the storeys above (and not below) the level of the podium.

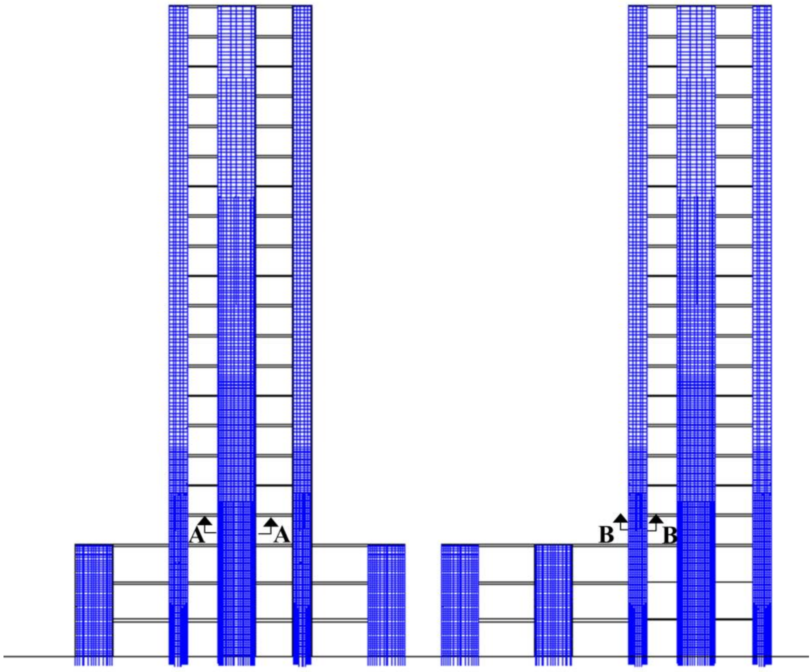


Figure 17: Elevation view of the buildings SB-1 and SB-2

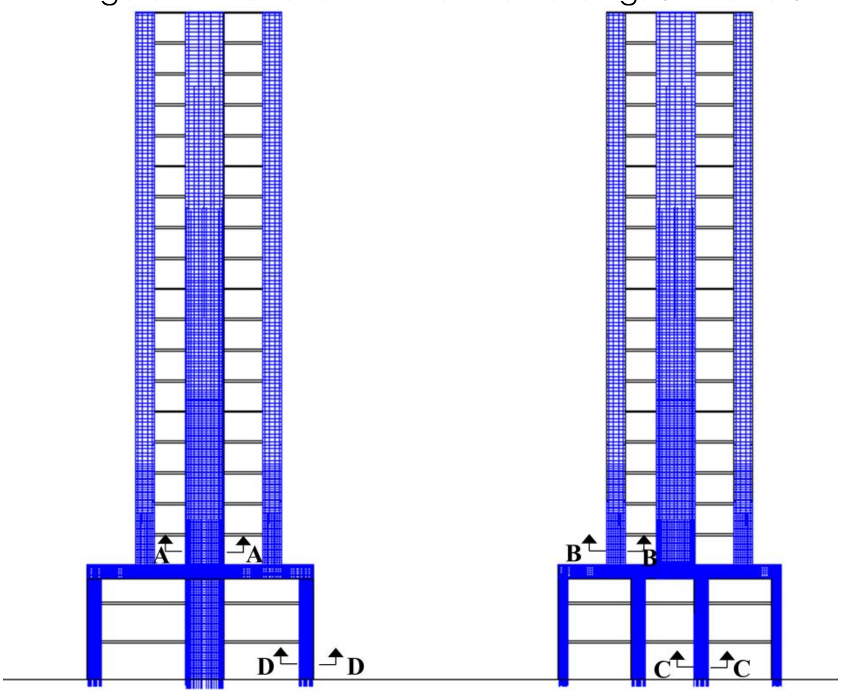


Figure 18: Elevation view of the buildings SB-1 and SB-2

Three performance levels have been considered in this study and these are the immediate occupancy (*IO*), Life safety (*LS*), and collapse prevention (*CP*) limit state. The limit for each level is presented in Table 4.

TABLE 4: PERFORMANCE LEVELS ADOPTED

Performance level	Limit
IO/First indication of yielding.	ISD corresponding to the first occurrence of flexural yielding in the RC walls making up the building (in the tower or the podium).
LS	Life safety limit state is defined as the ISD corresponding to: 1- Flexural yielding of all the tower walls above the podium interface level (or TFL) 2- Onset of nominal shear force capacity in the tower walls 3- Flexural yielding of the transfer plate (in building models TS-1 and TS-2) *Whichever occurs first*
CP	Collapse prevention limit state is defined as the ISD corresponding to 1. Onset of crushing compression strain in the confined core of the RC tower walls $\varepsilon_{cu} = -0.003$ 2. 50% loss of lateral strength in the tower walls (Walls 1 & 2) 3. Onset of nominal shear strength capacity of the central wall (ultimate strength) 4. Onset of ultimate tensile strain ($\varepsilon_{su} = 0.03$) in the reinforcement. *Whichever occurs first*

Fragility curves are developed following the Multiple Stripe Analysis (MSA) technique by maximising the likelihood for a limit state to be exceeded. Fragility curves for the four building models are presented in Figure 19.

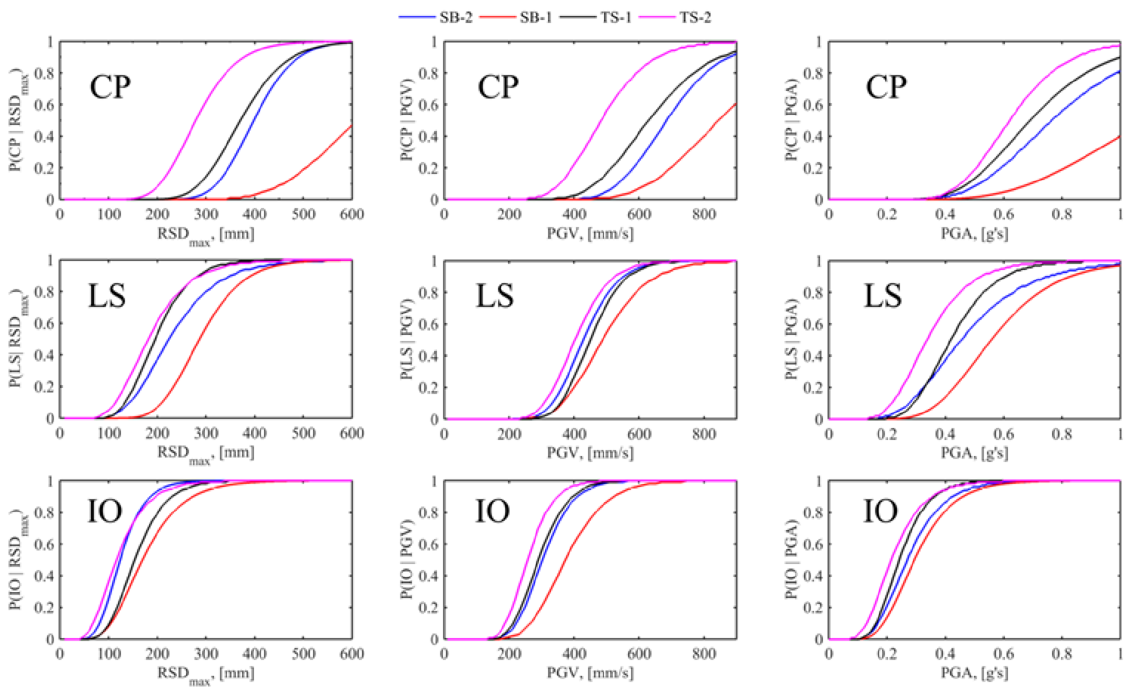


Figure 19: Fragility curves of the building models derived using RSDmax, PGV and PGA as the IM

ANALYTICAL DEVELOPMENT OF SEISMIC RETROFIT METHOD FOR LIMITED DUCTILE REINFORCED CONCRETE BEAM TO COLUMN JOINT USING DIAGONAL METALLIC HAUNCH

Analytical model has been developed for retrofitting RC exterior beam-column joint with single haunch element as shown In Figure 20. A single diagonal metallic haunch was proposed to reduce the shear demand at the exterior beam-column joint as Illustrated in Figure 21. Expressions have been developed to compute the shear demand at the joint based on the shear transferring factor, β. The β factor has been derived by Zabihi et al. (2016a; b) considering both beam and column deformations.

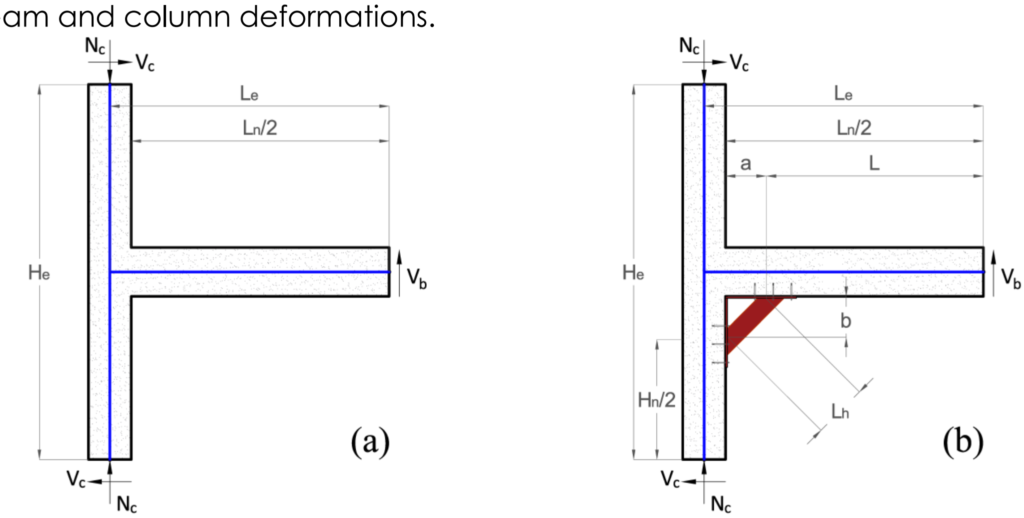


Figure 20: External actions on exterior beam-column joint: (a) Non-Retrofitted

System (NRS); and (b) Single Haunch Retrofitting System (SHRS).

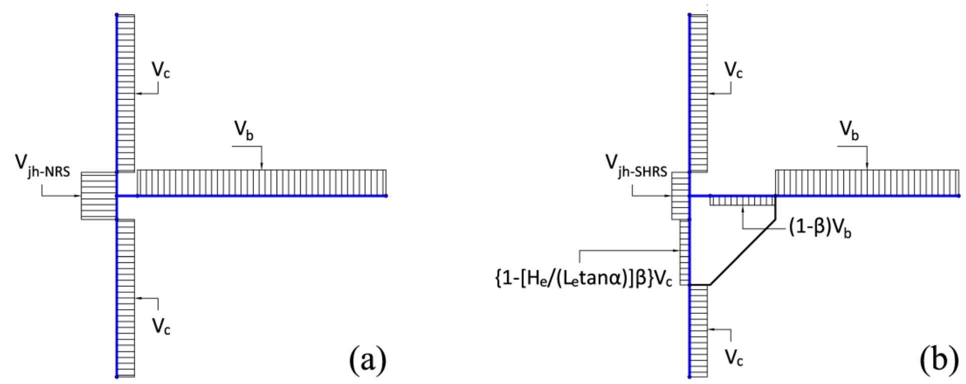


Figure 21: Shear force diagrams: (a) Non-Retrofitted System (NRS); and (b) Single Haunch Retrofitting System (SHRS).

A full scale three-storey RC moment resisting frame has been used as a case study. The frame has been designed based on the requirements in the 1980's (as shown in Figure 22). The frame is 9 m tall, 10 m wide, and is located on a deep or very soft soil site (i.e. Class D or E as defined in AS1170.4-2007) in Melbourne. The seismic weight was calculated by assuming 10 kPa gravity loads for all three levels including dead loads and 30% of imposed loads.

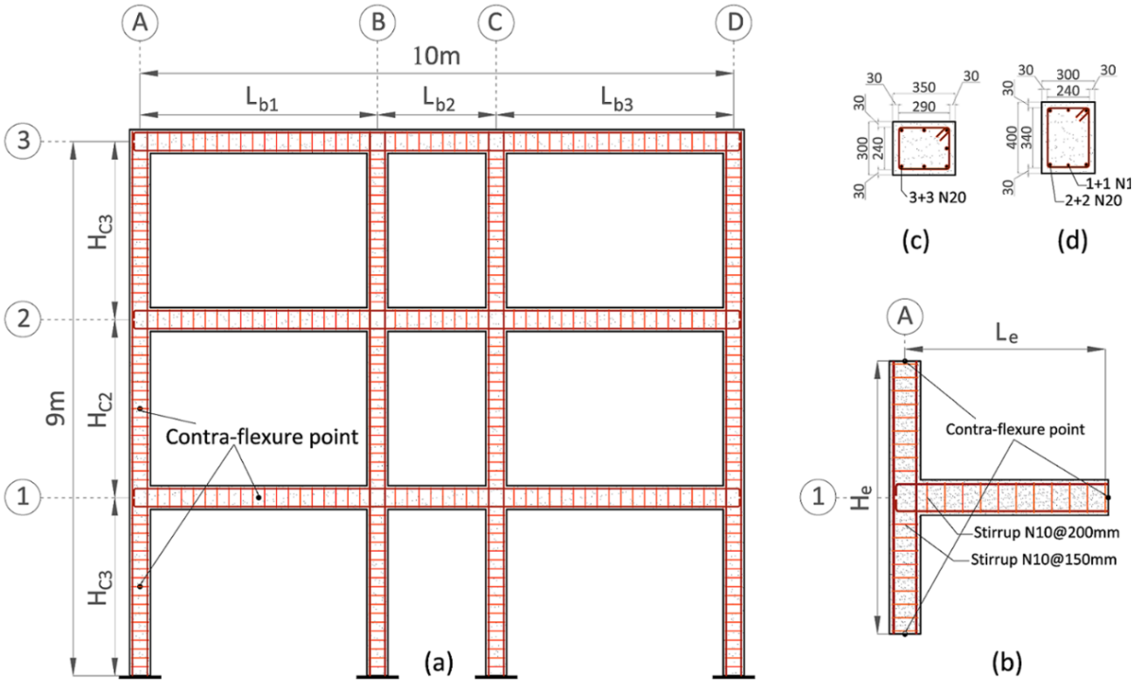


Figure 22: Geometry of case study model: (a) Full-scale RC moment resisting frame; (b) Exterior Beam-Column Joint; (c) Column section; (d) Beam section.

The limiting base shear force due to different failure mechanism is plotted in Figure 23 against the length of haunch. The non-retrofitted subassemblies (NRS) fails at a base shear level of 286 kN due to the formation of undesirable shear hinge at the joint zone. By applying a single diagonal haunch (SHRS) with 400 mm length and at an angle of 45 degrees to the beam, formation of the shear hinge is shifted from 286 kN base shear level to 339 kN. Although the retrofitted joint can resist against a stronger earthquake with 18% higher base shear force, the joint will still fail at the joint zone first which is considered undesirable from the perspective of capacity design principle. When the single diagonal haunch with the same angle but longer than 483 mm, a more favourable yielding mechanism, i.e. beam flexural yielding, will occur.

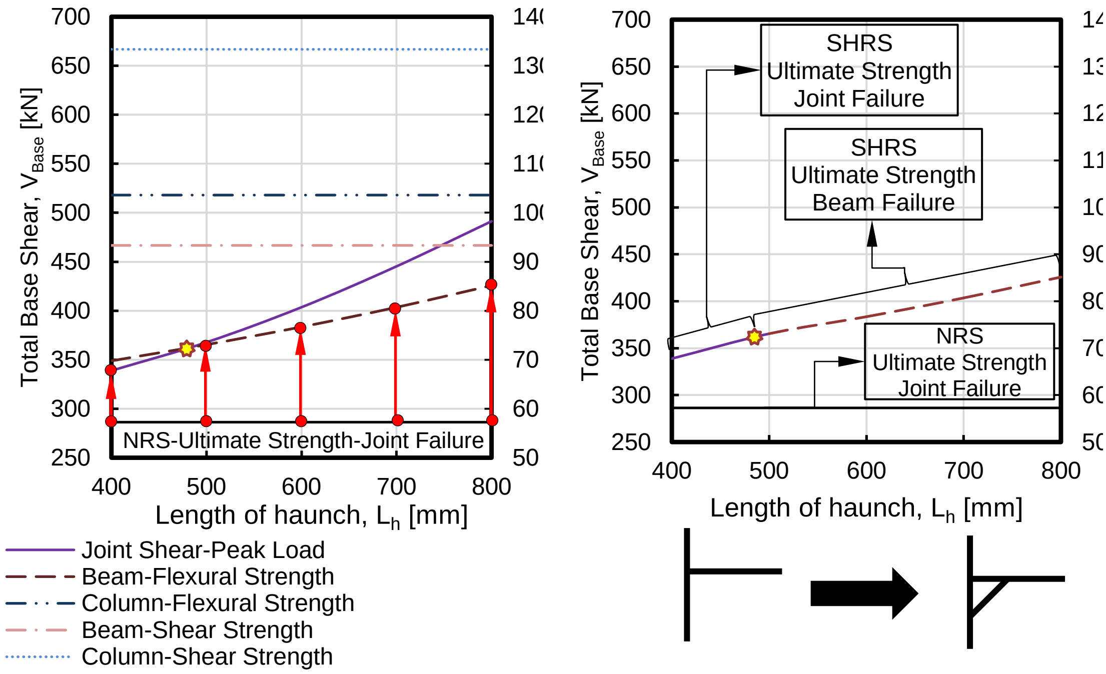


Figure 23: Strength hierarchy of the exterior beam-column joint subassembly before and after retrofit

ONGOING RESEARCH

A summary of the research undertaken over the previous year is outlined below.

- A team of delegates from Adelaide and Geoscience Australia traveled to York, WA to make progress on the End User project "Earthquake Mitigation Case Studies for WA Regional Towns". The activities included:
    - Meetings with Shire of York, York Society, York Business Association
    - Two public outreach sessions in York
    - Foot survey of York road bridges
    - Digitisation of council register of heritage listed buildings in York
    - RICS survey of York buildings (approximately 1830 buildings)

  - Foot survey of old non-residential URM buildings in York (approximately 50 buildings)
  - Foot survey of York businesses (approximately 75 businesses)
  - Detail survey of three buildings (St Patricks Church, Convent, Town Hall)
  - Digitisation of survey records (in progress)
  - Engagement with end users
- GA has prepared a building schema that categorises the Australian building stock into classes with distinctly different vulnerabilities to earthquake. The schema will enable the case studies undertaken later in the project to assign vulnerability functions developed by the project to buildings.
- Research in ongoing to produce URM building fragility curves for the non-structural URM building components. Significant research has been undertaken by University of Auckland by collecting empirical data from the 2010-2011 Canterbury earthquake swarm. This information will be supplemented by analyses of more scenarios to generate fragility curves that would be one of the deliverables in the second phase of Project A9.
- Experimental work into the durability of seismic retrofit of masonry elements is ongoing. FRP-strengthened specimens were subjected to environmental condition since the 2nd quarter of 2014-2015 and tested at different milestones of 6 months, 1 year, and 18 months. The last testing stage (24 months) is to be completed in the next few weeks.
- Experimental testing of high-strength RC columns under uni-directional and bi-directional cyclic load is on-going to validate numerically developed latera drift relationship for limited ductility reinforced concrete columns.
- Studies on different retrofitting options for limited ductility reinforced concrete buildings are ongoing.
- Preparation is well underway on experimental testing of axial stiffness of anchor groups in concrete as part of the haunch retrofit system for reinforced concrete beam-column joint. Numerical investigation is ongoing on the impact of haunch retrofit system on the seismic performance of limited ductile reinforced concrete frames.
- Numerical studies are currently underway to investigate the impact of the minimum requirement for the design hazard factor 0.08 g on the seismic design and performance of limited ductile reinforced concrete buildings. The potential impacts of designing buildings for lower annual probability of exceedance were also explored.

PROJECT REVISION – REVISED SCOPE AND GOING FORWARD

Earthquake Mitigation Case Studies for a WA Regional Town:

This End User is well engaged and scope revision has not been necessary. The project is well on track with significant desktop and site works already completed.

Results from site inspections and building typology study that were completed in in early 2018 are planned to be augmented to the building exposure data available from NEXIS in a follow-up desktop study. In the next few months, heritage-sensitive choices of seismic retrofit methods will be developed/formulated and costing that will provide the basis for cost-benefit analysis will be undertaken. End user demonstration of seismic retrofit methods is currently being planned on buildings that are scheduled for demolition.

Holistic Risk Assessment of Regulatory Requirements for Earthquake Design:

This proposed End User project did not go ahead.

Rapid Visual Screening (RVS) Procedure

This proposed End User project did not go ahead.

PUBLICATIONS LIST (JULY'17 - JUNE'18)

Note - only those published between 1 July 2017 & 30 June 2018 and has CRC in acknowledgments are listed.

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- Derakhshan, H., Nakamura, Y., Goldsworthy, H., Walsh, K.Q., Ingham, J.M., and Griffith, M.C. (2018). "Peak floor accelerations in unreinforced masonry buildings with flexible diaphragm", 10th Australasian Masonry Conference (10AMC), 11-14 February, Sydney.

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- Yunita Idris: FRP retrofit of non-ductile RC columns

University of Melbourne:

- Ryan Hoult: Seismic assessment of reinforced concrete walls in Australia
- Anita Amirsardari: Seismic assessment of reinforced concrete buildings in Australia including the response of gravity frames
- Mehair Yacoubian: Effects of Podium Interferences on Seismic Shear Demands in Tower Walls Supporting Buildings
- Alireza Mehdipanah: Buildings featuring irregularities in the gravity load carrying frames in low-to-moderate seismic regions
- Bin Xing: Seismic retrofit options for limited ductility RC buildings

Swinburne University:

- Scott Menegon: Seismic collapse behaviour of non-ductile RC walls
- Yassamin K Faiud Al-Ogaidi: FRP retrofit for non-ductile RC frames
- Alireza Zabihi: Seismic retrofit of RC beam-column joints
- Saim Raza: Collapse behavior of high-strength reinforced concrete columns in low to moderate seismic regions

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