Modelling 3D Limited-Ductile RC Frame Structures for Collapse Risk Assessment

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Abstract

Structures designed strictly in accordance to the required code of practice could still have a small probability of collapse in a major earthquake. This is the residual risk, which is unavoidable, and should be taken as a governing parameter for determining the performance goals of seismic design. Collapse assessment of structures involves combining information about the ground motion characteristics at the site with the nonlinear response behaviour of the structure. This can result in a collapse fragility curve, which describes the probability of collapse as a function of the ground motion intensity. The fragility curve can then be combined with the seismic hazard curve for computing the probability of collapse. Structures for these studies are often reduced to two-dimensional (2D) models that may not capture all the three-dimensional (3D) effects. This can influence the probability of collapse as both capacity and demands of structural components can potentially be under- or over-estimated. This paper presents the techniques for building 3D models for efficient collapse simulation and probabilistic risk assessment. To this end, the 3D model was used to predict sidesway-only collapse of a limited-ductile RC building subjected to a set of 38 uni-directional ground motions with increasing intensities. The incremental dynamic analysis results and the corresponding fragility curves are also presented.

Keywords: collapse assessment, three-dimensional, limited-ductile reinforced concrete, incremental dynamic analysis, fragility curve

1. INTRODUCTION

In seismic design of building structures, the risk of failure has to be limited to a very low level that is acceptable or tolerable by the public and stakeholders. In practice, however, such level of failure risk is not well-informed since structural design codes are usually prescriptive in nature, and engineers are only required to design and analyse the structure based on the stipulated loadings, without knowing the amount of residual risk.

Given the uncertainties associated with the estimates of seismic loads, and in addition to the unknown and uncertain response behaviours of structural elements in the nonlinear range, the modelling of structural collapse is highly complex and requires advanced modelling tools and is also computationally demanding. Hence, this topic has only become popular in the last decade, given various advancements in engineering and computational techniques (Lignos et al., 2011, Hashemi and Mosqueda, 2014).

Collapse modelling and the associated risk assessment have mainly been attempted by researchers in regions of higher seismicity. The types of buildings and seismicity patterns that were considered in those studies are primarily relevant to those regions, hence, it is questioned whether the research outcomes are applicable to regions of lower seismicity like Australia, where limited-ductile reinforced concrete (RC) buildings are prevailing.

In terms of structural modelling, whilst several studies on probabilistic collapse assessment of 2D non-ductile structures have been conducted (Liel and Deierlein, 2008, Celik and Ellingwood, 2010, Liel et al., 2011), few studies can be found on structures with the consideration of 3D effects such as biaxial bending or torsional effects (DeBock et al., 2014). This research takes up the challenge by developing the modelling techniques for evaluating the near-collapse nonlinear state of response of a 3D limited-ductile RC building in a probabilistic framework. To this end, the nonlinear incremental dynamic analysis (IDA) was conducted for the 3D 5-story, 5-bay RC building subjected to a set of 38 ground motions consisting of 19 recorded and 19 simulated uni-directional ground motions with increasing intensities. The collapse-level intensities of the model under all records were then assessed and the corresponding fragility curves were developed.

2. MODELLING TECHNIQUE FOR THE CASE STUDY BUILDING

Global collapse mechanism for most ductile structures and some limited-ductile structures with a weak or soft storey is governed by sidesway collapse. However, the collapse of limited-ductile concrete buildings is in fact more commonly controlled by the loss of support for gravity loads rather than the development of a sidesway collapse mechanism. Gravity-load collapse may be predicted by axial-load failure of columns, punching shear-failure of slab-column connections, or axial-load failure of beam-column joints.

The main purpose of this paper is to present a collapse assessment framework for 3D models while only flexural failure (i.e. sidesway collapse) has been taken into account in the modelling. A limited-ductile ordinary moment-resisting RC frame building (RC-OMRF) was designed as a case study based on the loading conditions in Melbourne. The 5-story, 5-bay structure with total height (H) of 21m was designed with identical span length of 8.4m in both directions and 4m story heights, except for the first story that has a 5m height. Two sections for the columns and four sections for the beams were considered. The beam and column sections are identical for all 5 stories. The design details are not presented in this paper but more details are presented in (Hashemi et al., 2014). The total gravity loads applied to the structure are 16.42MN for each floor and 15.8MN for the roof.

OpenSees (McKenna, 2011) was used to model the structure and perform nonlinear dynamic analysis. The inelastic flexural response of beam-column elements can be modelled using one of the five idealized model types shown in Fig.1. These inelastic models fall into two main categories: 1) lumped plasticity at the ends of the element, or 2) distributed plasticity along its length (Deierlein et al., 2010). In the concentrated plasticity models, the inelastic deformations are lumped at the ends of the element (Fig.1a & 1b). On the other hand, in the distributed plasticity models, the inelastic response is simulated either in a finite length hinge model (Fig.1c), or a fibre formulation (Fig.1d) that distributes plasticity by numerical integrations through the member cross sections and along the member length, or finally through the use of the finite element model (Fig.1e), which is the most complex model and breaks down the continuum along the member length and cross sections.

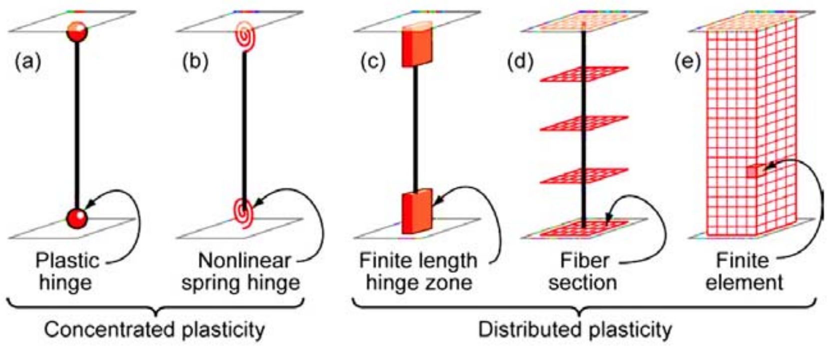


Figure 1. Idealized models of beam-column elements (after Deierlein et al., 2010)

Haselton et al. (2008) used the modelling type shown in Fig.1b to simulate the column response calibrated to 255 rectangular columns tests. The detailed hysteretic nonlinear model representing the rotational springs is based on regression-based equations which were developed for estimating the linear and nonlinear parameters according to column properties. In-cycle and cyclic degradation are included in the model and are defined by the regression-based equations.

Most current modelling approaches for RC structures are based on lumped plasticity (Liel et al., 2011, Shoraka et al., 2013) that requires assembling of three elements for the 2D model. This will increase to 5 elements for each individual beam-column element in a 3D model, considering only the flexural failure. Further enhancing the model to other modes of failure could lead to a multifaceted and complex model. In order to avoid this, the *BeamWithHinges* element, shown in Fig.1c, was used that specifies only one element for each beam-column element. Explicit modelling of beam-column joints and bar-slip effects were not included since they are considered not necessary for performing collapse modelling of RC frames, according to FEMA-P695 (FEMA, 2009).

The modelling parameters of each beam-column element were predicted using empirical equations developed by Haselton et al. (2008) that relate the design parameters to the peak-ordinated hysteresis response (shear-drift) of the RC column. The modelling parameters are based on the modified Ibarra-Medina-Krawinkler (IMK) deterioration model (Ibarra and Krawinkler, 2005) for the flexural behaviour. The moment-curvature properties of the nonlinear sections in *BeamWithHinges* elements were calibrated to eventually match the shear-drift hysteretic response of the beam-column elements. Fig.2 illustrated the procedure of calibrating the numerical elements.

After developing the numerical model, the elastic fundamental period of vibration (T_1=0.887sec) and the corresponding first mode shape were obtained through eigenvalue analysis. A nonlinear static pushover analysis was then performed with the lateral force distribution that is proportional to the first mode until exceeding the point of 20% strength loss. Second-order effects (the global P-Δ) were considered in the modelling and analysis processes. The results of the pushover analysis as shown in Figure 3 illustrate that most of the energy dissipation occurs in the lower stories.

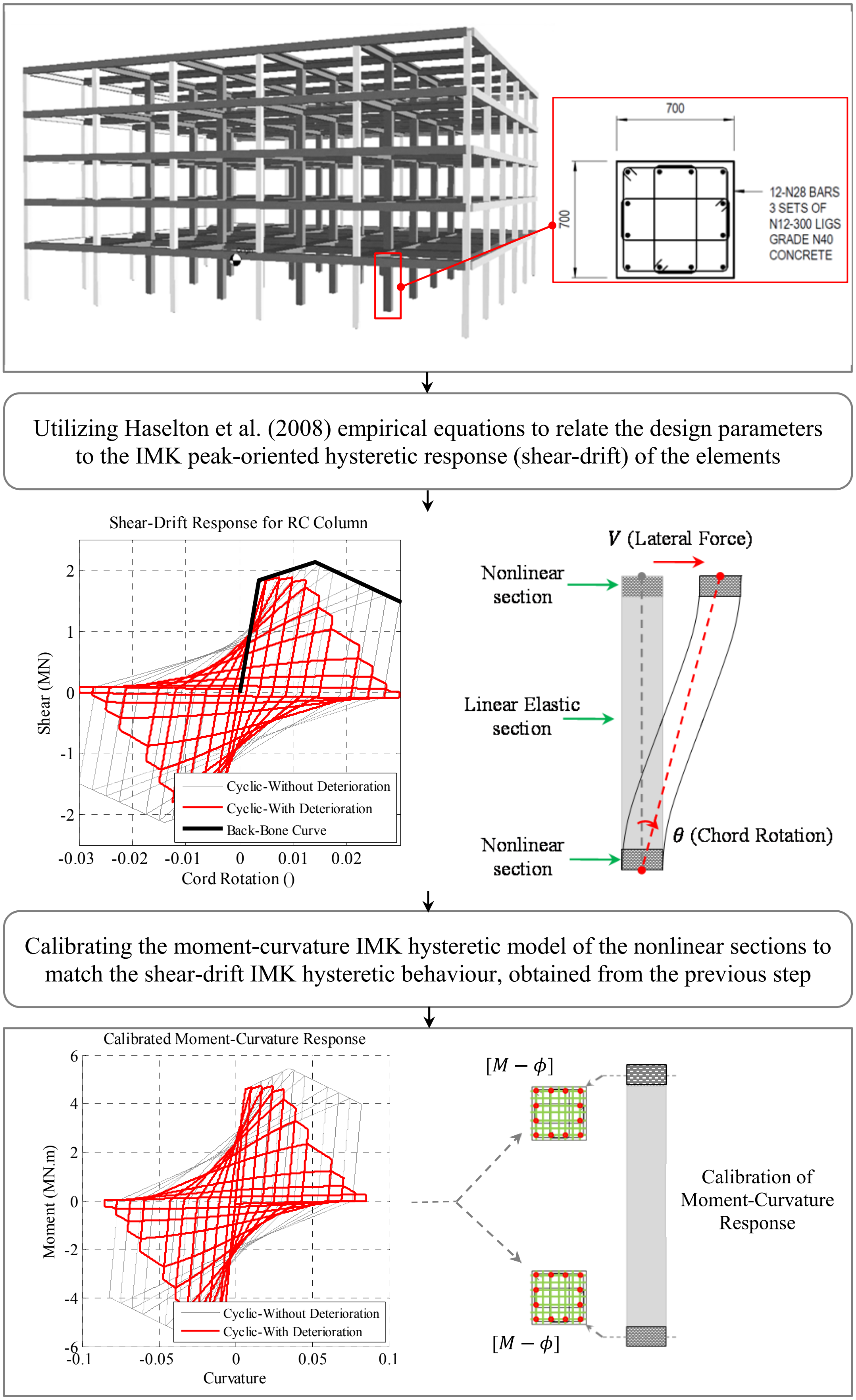
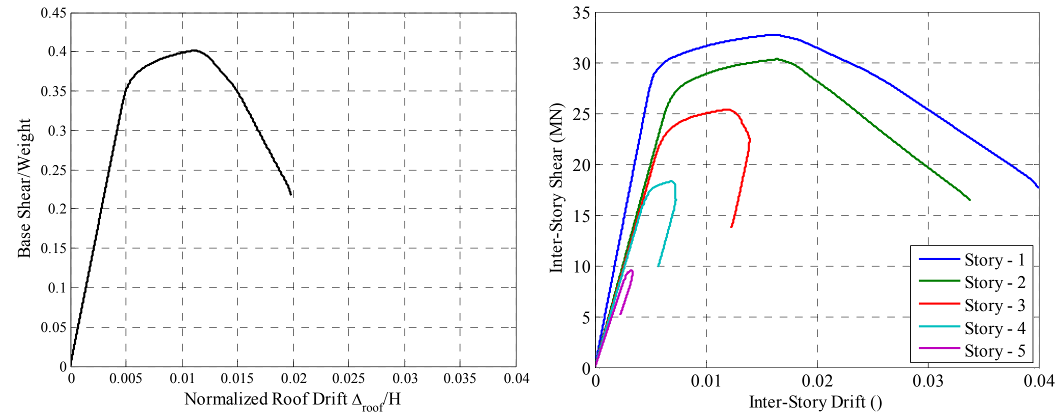


Figure 2. Calibration of the numerical model



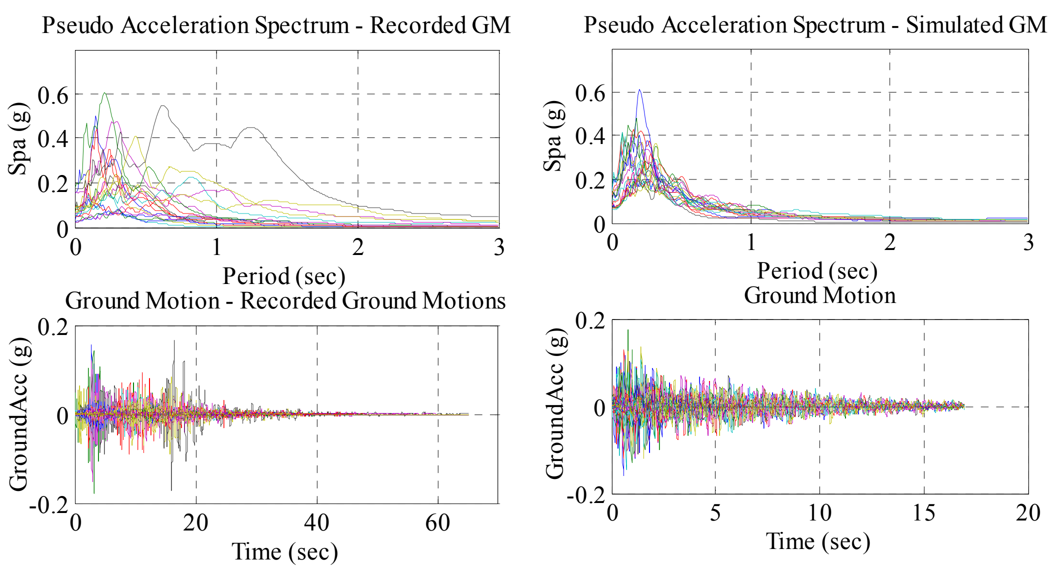
a) Overall response b) Inter-story response

Figure 3. Nonlinear pushover analysis

3. COLLAPSE ASSESSMENT PROCEDURE

3.1. Ground Motion Inputs

In order to obtain a range of possible structural response behaviours, four representative earthquake scenarios, with magnitude (M) and source-to-site distance (R) of M5.5R17, M6.0R28, M6.5R40 and M7.0R90, have been considered based on the 1-in-500-year hazard level of rock sites in Melbourne (i.e. $Sv_{,max}$=110mm/s). A suite of 19 recorded ground motions was selected from the PEER database (http://peer.berkeley.edu/nga/), whilst a complementary set of 19 synthetic ground motions was generated using stochastic simulations based on the seismological model (Lam et al., 2000, Lam et al., 2010). The pseudo acceleration (Spa) spectrum and acceleration time histories of all the input ground motions are shown in Fig.4.



a) Recorded ground motions b) Simulated ground motions

Figure 4. Ground motions selected for case study

3.2. Collapse indicator

The collapse indicator is commonly defined from the response parameters of the building. Considering only the flexural failure, maximum degradation in inter-story shear resistance (i.e. developing a weak story) and/or maximum inter-story drift ratio (i.e. developing a soft story) can be used as the collapse indicator. In this study, a conservative value of 2% maximum inter-story drift was selected for the sidesway collapse criterion of the case study building. This was selected by considering 20% maximum degradation in inter-story shear resistance as obtained from the pushover analysis. Vertical collapse mechanisms, which are not directly simulated in the structural model, are not considered in this assessment.

3.3. Fragility Analysis

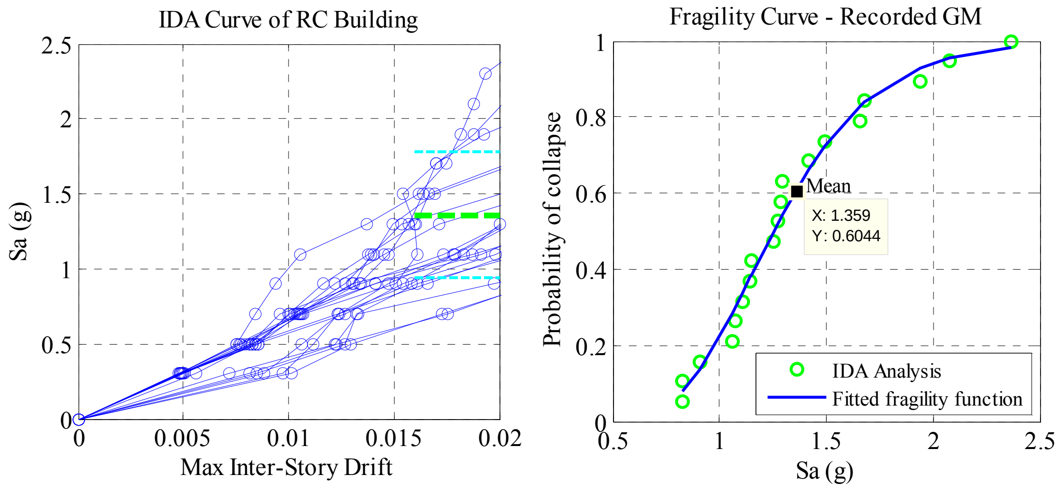
The incremental dynamic analysis (IDA) was conducted to organise nonlinear dynamic collapse analysis of the RC-OMRF model subjected to 38 ground motions (Vamvatsikos and Cornell, 2002). For the nonlinear simulation, Rayleigh damping based on initial stiffness is used, and damping ratios at the first and third mode are set to 5%. Each uni-directional ground motion is individually applied to the 3D frame model and amplitude scaled according to the spectral acceleration at the first-mode period $Sa(T_1)$. The ground motions are increasingly scaled until reaching the collapse state of the building. The outcome of this assessment is a structural collapse fragility function, which is a lognormal distribution relating the structure's probability of collapse to the ground-motion intensity.

Figures 5, 6 and 7 present the results of nonlinear incremental time-history analyses and the associated fragility curves for the recorded, simulated and combined ground motions respectively. The median collapse capacity in terms of $Sa(T_1)$ is around 1.47g, provided that the simulation is restricted to sidesway-only collapse case with an inter-story drift limit of 2%, which has taken into account the strength degradation in flexural hinges of beam-column elements.

However, observed earthquake damage and laboratory studies have shown that shear failure and subsequent loss of gravity-load-bearing capacity in one column could lead to progressive collapse in limited-ductile RC frames. Although column shear failure is not incorporated directly because of the difficulties in accurately simulating shear or flexure-shear failure and subsequent loss of axial load-carrying capacity (Elwood, 2004), the simplified approach implemented by Liel et al. (2011) can be used to modify the fragility curve in order to take the uncertainties related to the axial-load failure into account. In addition, uni-directional loading will lead to the underestimation of both capacity of the columns and demands on the columns. Therefore, to capture the effects of bidirectional loading, torsion and floor diaphragm flexibility in collapse assessments, the study should be extended to bi-directional loading and real 3D situations.

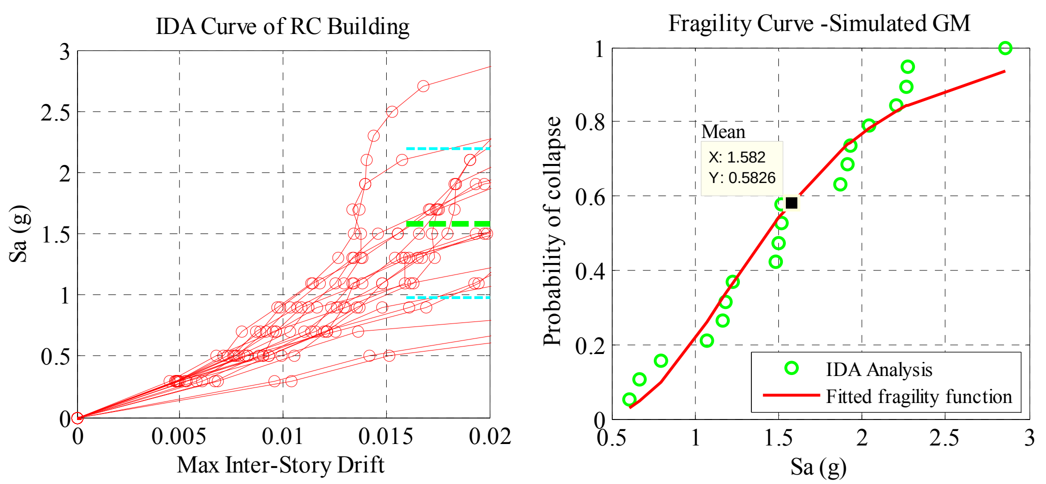
4. SUMMARY AND CONCLUSION

Collapse performance assessment of a three-dimensional 5-story, 5-bay limited-ductile ordinary moment-resisting reinforced concrete structure was presented. The modelling techniques were discussed to efficiently construct the numerical model of a RC structure for nonlinear time history analysis. The structural model was then subjected to incremental dynamic analysis to quantify the state of damage with respect to the ground motion intensity level. A set of 38 ground motions including recorded and simulated ground motion records was used to propagate the effects of uncertainties in ground motions on the response of structures. The outcome of the analysis was used to build the fragility curves, which show the probability of structural collapse given a certain level of ground motion intensity. The simulation considers only sidesway collapse that occurs typically in limited-ductile RC structures with a soft or weak story. In future studies, the framework will be extended to bi-directional loading to consider real three-dimensional situations. The structural model will be extended to include additional failure modes, namely sidesway/axial collapse mechanism.



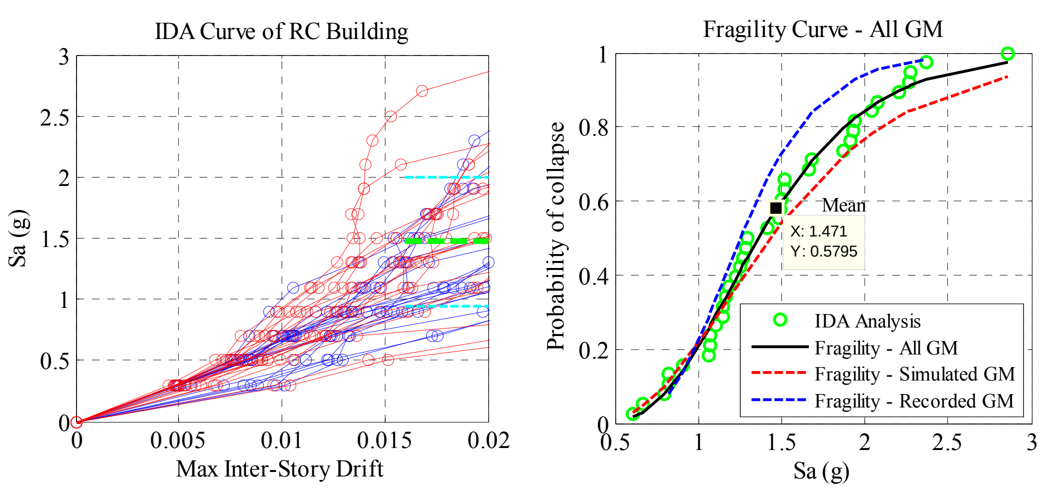
a) Results of the IDA analysis b) Collapse fragility

Figure 5. Summary of the results for the recorded ground motions



a) Results of the IDA analysis b) Collapse fragility

Figure 6. Summary of the results for the simulated ground motions



a) Results of the IDA analysis b) Collapse fragility

Figure 7. Summary of the results for all the ground motions

ACKNOWLEDGEMENT

The authors gratefully acknowledge Elisa Lumantarna and Nelson Lam of The University of Melbourne for their continuous partnership and collaboration on earthquake ground motion characterisation for regions of low-to-moderate seismicity, from which the input ground motion data has been adopted in this study.

REFERENCES

Celik, O. C. & Ellingwood, B. R. (2010) Seismic fragilities for non-ductile reinforced concrete frames - Role of aleatoric and epistemic uncertainties. Structural Safety 32(1):1-12.

Debock, D. J., Liel, A. B., Haselton, C. B., Hooper, J. D. & Henige, R. A. (2014) Importance of seismic design accidental torsion requirements for building collapse capacity. Earthquake Engineering & Structural Dynamics 43(6):831-850.

Deierlein, G. G., Reinhorn, A. M. & Willford, M. R. (2010) Program Plan for the Development of Collapse Assessment and Mitigation Strategies for Existing Reinforced Concrete Buildings. Report NIST GCR 10-917-7.

Elwood, K. J. (2004) Modelling failures in existing reinforced concrete columns. Canadian Journal of Civil Engineering 31(5):846-859.

FEMA (2009) Quantification of Building Seismic Performance Factors. Washington, D.C., USA, Report FEMA P695.

Haselton, C. B., Liel, A. B., Lange, S. T. & Deierlein, G. G. (2008) Beam-column element model calibrated for predicting flexural response leading to global collapse of RC frame buildings. Pacific Earthquake Engineering Research Center.

Hashemi, M. J., Al-Ogaidi, Y. K. F., Wilson, J. & Abdouka, K. (2014) Collapse Simulation of Multi-Story RC Buildings through Hybrid Testing. In: Australian Earthquake Engineering Society Conference,Lorne, Victoria.

Hashemi, M. J. & Mosqueda, G. (2014) Innovative substructuring technique for hybrid simulation of multistory buildings through collapse. Earthquake Engineering & Structural Dynamics 43(14):2059-2074.

Ibarra, L. F. & Krawinkler, H. (2005) Global collapse of frame structures under seismic excitations. The John A. Blume Earthquake Engineering Center, Report TB 152.

Lam, N., Wilson, J. & Hutchinson, G. (2000) Generation of synthetic earthquake accelerograms using seismological modelling: A review. Journal of Earthquake Engineering 4(3):321-354.

Lam, N., Wilson, J. & Tsang, H. H. (2010) Modelling earthquake ground motions by stochastic method. In: Stochastic Control. (Myers, C. (ed)) Sciyo, pp. 475-492.

Liel, A. B. & Deierlein, G. G. (2008) Assessing the Collapse Risk of California's Existing Reinforced Concrete Frame Structures: Metrics for Seismic Safety Decisions. The John A. Blume Earthquake Engineering Center, Report TR No. 166.

Liel, A. B., Haselton, C. B. & Deierlein, G. G. (2011) Seismic Collapse Safety of Reinforced Concrete Buildings. II: Comparative Assessment of Nonductile and Ductile Moment Frames. Journal of Structural Engineering 137(4):492-502.

Lignos, D. G., Krawinkler, H. & Whittaker, A. S. (2011) Prediction and validation of sidesway collapse of two scale models of a 4-story steel moment frame. Earthquake Engineering & Structural Dynamics 40(7):807-825.

Mckenna, F. (2011) OpenSees: A Framework for Earthquake Engineering Simulation. Computing in Science & Engineering 13(4):58-66.

Shoraka, M. B., Yang, T. Y. & Elwood, K. J. (2013) Seismic loss estimation of non-ductile reinforced concrete buildings. Earthquake Engineering & Structural Dynamics 42(2):297-310.

Vamvatsikos, D. & Cornell, C. A. (2002) Incremental dynamic analysis. Earthquake Engineering & Structural Dynamics 31(3):491-514.