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FRAGILITY CURVES FOR NEWZEALAND BUILDINGS WITH REFLECTIONS FROM THE CANTERBURY EARTHQUAKE SEQUENCE

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Abstract

Fragility functions establish relationship between ground motion intensity and damage and they are collectively an important component in developing tools for seismic risk assessments. These functions can be derived by various methods supported by empirical data, expert opinion, and analytical models. In this study, a procedure based on displacement-based earthquake loss assessment (DBELA) is adopted to derive fragility functions for population of buildings. Focus is placed on concrete buildings particularly for the reason, in New Zealand cities, most of the non-residential buildings are of concrete construction. The characteristics of New Zealand buildings were exclusively modeled and not derived from models developed overseas as they are significantly different from New Zealand building stock. The method follows a probabilistic displacement-based approach that takes into account various sources of uncertainties in geometrical characteristics, deformation capacities of buildings and ground motion demands in deriving fragility functions. Spectral displacement demands from ground motions are modeled based on probabilistic seismic hazard analyses for a high seismicity region in New Zealand and real earthquake records from recent events. The probabilities of failure at different limits states are determined at various intensity levels of shaking and a least square regression method is adopted to fit cumulative lognormal distribution curves as fragility functions.

In addition, a mapping scheme is proposed to translate the proportions of building damage distribution derived from fragility functions at a given intensity of shaking in terms of color tagging that is commonly used in post-event building safety evaluations. An attempt was made to calibrate the mapping scheme based on the observed damage statistics during Christchurch 2011 earthquake. The proposed mapping scheme can be used to predict with reasonable confidence the likely distribution of building damage (in terms of the commonly used damage states and/or color tags) in future earthquakes and will be useful in planning emergency response after an earthquake.

Keywords: building safety evaluation, concrete frame buildings; displacement-based method; fragility functions;

1. Introduction

Seismic loss estimation methodologies require fragility functions to relate physical damage to ground motion shaking intensity. Fragility functions represent probabilities of exceeding different limit states of building response and being within certain damage states for a given shaking intensity level. A large number of methods for fragility assessment have been proposed in the past. The authors acknowledge exhaustive past research work reported in literature and cite a few to refer to the three well known methods: (i) empirical method using field data [1,2,3]; (ii) expert-judgment method [4,5]; and (iii) numerical analysis method [6,7,8]. Even though, the empirical fragility curves are the most realistic because any field observational damage data is very valuable as it gives truthful information on building performance, historical earthquake events with large magnitudes that caused building damage are infrequent and the data is scarce. Therefore, the analytical and expert-judgment methods are recognized as better alternatives. However, when empirical data become available, analytical predictions can be verified with empirical damage data as demonstrated in this study.

In order to derive fragility functions for a building class, the damage potential or the vulnerability of a typical building representing that class needs to be assessed at various intensity levels, so that a continuous function can be established. Displacement-based earthquake loss assessment (DBELA) method provides estimation of building damage distributions for a given level of shaking intensity. The procedure adopts a probabilistic framework that takes into account various sources of uncertainties in geometrical characteristics, deformation capacities of building and ground motion demands. In this study, the DBELA procedure, originally proposed by Crowley et al.,[9] and subsequently modified by the author [10] is adopted.

In the previous study [10], first, the modified displacement-based vulnerability method was evaluated by comparing the estimated building damage distribution for Christchurch building stock in the event of February 2011 Christchurch earthquake with the observed damage data. The study used a set of building models that were exclusive representatives of New Zealand buildings. The densely recorded ground motions and systematically conducted building tagging exercise conducted for building safety evaluation in Christchurch provided an opportunity to validate the method to estimate the vulnerability in terms of probabilities of exceeding different damage limit states for a given shaking intensity.

The present study builds on the previous study [10] and aims to derive fragility functions that represent probabilities of exceedence of damage limit states with varying shaking intensity. Recently, a methodology to extend DBELA method to derive fragility functions was proposed [11] in the form of a 'fragility calculator'. The present study incorporates some principles suggested in [11], while efforts have been taken to adopt New Zealand specific building models and different forms of representing New Zealand specific ground motion demands at varying levels of intensities. The procedure is illustrated through four building categories, viz., low and medium-rise reinforced concrete buildings of pre and post 1976 construction era that were also used to evaluate the DBELA methodology under Christchurch event. The models take into account various sources of uncertainties in geometrical characteristics, deformation capacities of buildings.

The DBELA methodology considers ground motion demands in terms of spectral displacement. There are different approaches by which the ground motion demands for a region can be modelled. A few approaches include: (i) considering earthquake scenarios for combinations of magnitude and distance where attenuation models provide estimates of spectral demands and their associated uncertainties; (ii) performing probabilistic seismic hazard analyses (PSHA) for the region where the spectral values have already accounted for uncertainty due to randomness associated with earthquake events or record-to-record variability[12]; (iii) using real accelerograms that were recorded in the region of interest, if they are available; and (iv) in the absence of real records, using ensemble of accelerograms to account for record-to-record variability. Note that there is no well-established procedure yet available to choose the right representative accelerograms for deriving fragility functions [11] and many schools of thoughts are out there around scaling procedure that attempts to preserve the original characteristics of acceleration records. The focus of the current study is to develop fragility functions for specific New Zealand building types located in high and moderate seismic regions for all possible ground motion scenarios affecting the regions. Therefore ground motion demands based on PSHA and two recent real earthquake records that affected the regions of interest are used in this study.

As mentioned earlier, vulnerability assessment is an important component of loss modelling as they provide proportions of buildings in different damage states. Another application was demonstrated in the previous study through a mapping scheme [10] to translate the probabilities of damage states into a measure of color tagging as a part of building safety evaluation for occupancy after an earthquake. Information on proportions of buildings in various color tagging will be useful for emergency planning purposes post an earthquake event. The parameters of the mapping scheme were calibrated using the data of observed building damage from the Christchurch event [10].

In this study, the above application is extended to refine the parameters of the mapping scheme by comparing the damage states provided by the new fragility curves at an intensity corresponding to Christchurch event and the observed damage data. The refined mapping scheme can be used with reasonable confidence in estimating the number of buildings in different color tagging groups for emergency planning purposes.

Based on the above discussion, the two major objectives of the present work can be stated as: (i) developing fragility curves for a few select types of New Zealand buildings; and (ii) proposing new refined mapping scheme to translate damage state probabilities into color tagging groups. The study includes:

(i) Simulation of building characteristics for reinforced concrete frame buildings [10] considering the uncertainties in their geometrical and mechanical properties;

(ii) Considering ground motion spectral demands for a given site class based on probabilistic seismic hazard assessment (PSHA) studies on a high seismicity region (Wellington) to have conservative estimates of probabilities of failure of buildings; and from real ground motion records: (a) the February 2011 Christchurch earthquake; (b) July 2013 Cook Strait earthquake in Wellington to constrain the analytical predictions and to verify the estimates with observed damage of building stock.

(iii) Determination of probabilities of exceedence of various damage limit states for all the scenarios described in step (ii) and generation of fragility curves by fitting lognormal distribution curves to the list of cumulative probabilities of damage exceedence versus intensity measure levels by using the mean least squares regression method; and

(iv) Proposing a refined calibrated mapping scheme to estimate proportions of a regional building stock in different color tagging groups at a given level of shaking intensity.

2. Review of Displacement-based method

Several research works on vulnerability assessment have adopted displacement-based approach [13,14]. A probabilistic displacement-based vulnerability assessment framework originally proposed by Pinho et al. [15] and subsequently improved with several modifications by Uma et al[10] is adopted in this study. The approach is simple and practical to deal with variability in building parameters and uncertainty in ground motion demands. A brief summary of the procedure is given below.

The methodology considers every building class represented by a typical building which is modeled as an equivalent single degree of freedom (SDOF) oscillator. A population of buildings representative of the building class is generated using a Monte Carlo approach with the range of structural properties representing reinforced concrete frame buildings in New Zealand. In every simulation, randomly assigned geometrical and mechanical properties are used to obtain the initial parameters (initial period and yield displacement) of the SDOF models from which the logarithmic mean (i.e. median) and logarithmic standard deviation (i.e. dispersion) of the initial parameters are arrived for the representative population. Within a probabilistic framework, the limit states (LS) are defined in terms median values of building displacement (or the drift ratio of the SDOF oscillator) and considering uncertainties. The effective periods corresponding to the predetermined limit states are determined based on the initial period and the ductility realized at the drift limit states. Next, spectral displacement demands at those effective periods are obtained from the over-damped ground motion spectral demands. Then damage state probabilities are estimated by comparing the displacement realized by the building and the earthquake demand considering various sources of uncertainties related to the building and ground motion parameters.

3. Modeling of building characteristics

The focus is placed on concrete buildings particularly for the reason, in New Zealand cities, most of the non-residential buildings within the central business districts are of concrete construction. The central business districts within high and moderate seismic regions (e.g. Wellington, Christchurch) comprise many low (1 to 3 storeys) and medium-rise (4 to 7 storeys) buildings and a few very high (7+ storeys) concrete moment resisting and dual wall-frame buildings. In this study, focus is given to concrete moment resisting frame buildings constructed prior to year 1976 are considered to be with limited ductility and have potential to fail in column sway mechanism, whereas those buildings constructed after 1976 are considered to be ductile and follows a beam sway mechanism in its inelastic response. Totally four building classes of reinforced concrete moment resisting frames are considered in this study: (i) low-rise Pre 1976; (ii) low-rise 1976 on; (iii) medium-rise pre 1976; and (iv) medium-rise 1976 on.

3.1. Derivation of initial parameters for building classes

For each building class, a Monte Carlo procedure is adopted to generate a large set of values for the yield displacement D_y, at the effective height of the SDOF system and its initial period T_y and they are obtained using randomly generated values of geometrical and material properties to cover every possible building within that class. Typical variables include storey height, number of stories, beam and column dimensions, yield strain and tensile strength of steel and compressive strength of concrete. The range of values adopted for the considered building classes are specified elsewhere [10] and it is believed that they represent the properties of buildings in high and moderate seismicity regions. The displacement capacity at yield is computed at the effective height of the SDOF system using established empirical relationships using the expressions proposed by Glaister and Pinho [16] for the appropriate building groups. The initial period of every building randomly generated (i.e. corresponding to the elastic stiffness) is computed based on the empirical equation as referred by the commentary of the NZ standard NZS 1170.5: 2004. The generated data are assumed to be log-normally distributed, the median and dispersion of D_y and T_y (i.e. $median\_D_y$, $median\_T_y$, β_D_y and β_T_y as noted in the following sections) are reported in Table 1[10].

Table 1 – Initial properties of building classes [10]

	Yield displacement, m		Initial period, s		Effective height, m	
	Median	**Dispersion**	**Median**	**Dispersion**	**Median**	**Dispersion**
Concrete Frames (1976 on) (ductile)						
Low-rise	0.027	0.33	0.5	0.3	4.4	0.31
Medium-rise	0.074	0.21	1.1	0.24	12.1	0.15
Concrete Frames (Pre 1976) (limited ductile)						
Low-rise	0.026	0.33	0.5	0.3	4.4	0.31
Medium-rise	0.065	0.21	1.1	0.24	12.1	0.15

3.2. Definition of limit states and displacement capacities at inelastic limit states

Four limit states (LSi, i=1,4) are considered in terms of maximum drift ratio at the effective height of an equivalent SDOF model. LS1 denotes the 'significant yield point' in the bilinear curve which is marked well beyond the first onset of yield and by then, the drift sensitive non-structural elements might have got damaged. LS3 is considered at a drift limit corresponding to the ultimate limit state satisfying the life-safety criteria. LS4 is considered to be at the drift limit corresponding to collapse prevention criteria. LS2 is defined mid-way between LS1 and LS3.

The median drift ratios at the effective height for the four limit states that define the displacement capacities ($median\_D_{LSi}$) are listed along with the dispersions (β_D_{LSi}) in Table 2.

Table 2 – Median threshold drift ratios and dispersions considered for the frame buildings [10]

	LS1		**LS2**		**LS3**		**LS4**	
	Med.	**Disp.**	**Med.**	**Disp.**	**Med.**	**Disp.**	**Med.**	**Disp.**
Post-1976	0.6%	0.25	1.5%	0.35	2.5%	0.4	4%	0.45
Pre-1976	0.5%	0.25	1%	0.3	1.5%	0.35	2.5%	0.4

4. Modeling of ground motion demands

Traditionally, large sets of ground motion accelerations are used [11] to derive the spectral displacement demands in order to capture the effect of record to record variability on structural response. In this context, GeoNet (www.geonet.org.nz) in New Zealand provides archives of ground motion records. However, as explained before in section 1, there is no known formal study carried out to choose the right set of accelerograms that are suitable for deriving fragility functions for New Zealand buildings. Among all the four approaches described above, the present study adopts two approaches to model the ground motion spectral demands for a site class of 'deep soil':

(i) based on probabilistic seismic hazard analyses (PSHA) as they are the best option as the spectra represent the envelope of the demands from various possible scenarios that could affect the region. They are (a) recommended design spectra from the design standards NZ 1170.5:2004[17] for a high seismicity region in New Zealand (Wellington); and (b) site specific design spectra developed for a location within central business district of Wellington where is densely populated by low-rise and medium-rise buildings. A set of smooth design spectra were generated for different return periods (25 years, 500 years, 1000 years and 2500 years) that represent the median. Note that the spectral demands have already accounted for aleotary (record-to-record) uncertainty.

(ii) by considering real records from the recent events that affected buildings in urban areas. For example, two recent events, namely, Christchurch February, 2011 for its high shaking intensity severely damaged buildings in the central business district (CBD) of Christchurch; and the Cook Strait event in July 2013 that was moderate and resulted in slight to moderate damage to buildings in Wellington CBD. It is proposed that the damage probabilities from the real events would help to constrain the fragility curves; and any observed damage data/information from those events will be helpful to calibrate the curves.

Note that the geometric mean of the spectral demands from the strong motion records of four stations closely located in around Christchurch CBD as considered in the previous study [10] is used in the current study. The ground motion recorded at a strong motion station within Wellington CBD during the Cook Strait event is used in this study.

4.1. Derivation of spectral displacement demands at inelastic limit states

Spectral displacement demands are obtained at the equivalent period at inelastic limit states. The procedure as explained in [10] is reproduced here and explained with reference to the ground motion cases considered in this study. Note that in the displacement-based approach, the median equivalent period $median\_T_{LSi}$ is based on the initial median period $(median\_T_y)$ and the ductility realized at a chosen limit state $(D_{LSi})$and the corresponding dispersion β_T_{LSi} are determined as per Eq. (1) and Eq. (2):

$$median\_T_{LSi} = \sqrt{\left(median\_D_{LSi} / median\_D_y\right)} \cdot median\_T_y \tag{1}$$

$$\beta\_T_{LSi} = \sqrt{0.5\left(\left(\beta\_D_{LSi}\right)^2 - \left(\beta\_D_y\right)^2\right) + \left(\beta\_T_y\right)^2} \tag{2}$$

The median spectral displacement demand at a limit state ($median\_S_{d(demand)LSi}$) is directly obtained from the spectral acceleration at the median period corresponding to that limit state ($median\_T_{LSi}$) obtained from the elastic demand spectrum using Eq. (3).

$$median\_S_{d(demand)LSi} = \psi_{eff}\, S_a\left(median\_T_{LSi}\right) / \left(2\pi / median\_T_{LSi}\right)^2 \tag{3}$$

The elastic spectral displacement demands are reduced using spectra reduction factor ψ_{eff} to account for the increased damping due to inelastic response at various limit states (except for LS1). The spectra reduction factor ψ_{eff} is a function of equivalent viscous damping ratio which is obtained using the ductility level achieved at a given limit state and the damping reduction factor for a given structural system [18].

It is recognized that the spectral acceleration demands are dependent on the period and this dependency will influence the dispersion of spectral displacement demand. However, the spectral demands arrived based on PSHA procedure have already accounted for the aleotary uncertainty (in other words, record-to-record variability). However, there are some uncertainty still needs to be considered for the influence of period on spectral demand. Similarly, when dealing with 'actual' recorded ground motions from real events, it can be reasonably assumed that the dispersion in the elastic spectral displacement demand is contributed mainly by the dispersion of period (β_T_{LSi}). Assuming the spectral displacement demand is lognormally distributed, its dispersion can be calculated as in Eq. (4).

$$\beta\_S_{d(demand)LSi} = \sqrt{2}\,\beta\_T_{LSi} \tag{4}$$

5. Generation of fragility curves

Once the spectral displacement capacities and demands are obtained for every limit state period considered, the probabilities of exceedence of limit states (LS1 to LS4) are determined using cumulative distribution function of the standard normal distribution (Φ) from Eq. (5) and Eq. (6).

$$Z = \left| \frac{\ln\left(median\_S_{d(demand)LSi} / median\_D_{LSi}\right)}{\sqrt{\left(\beta\_D_{LSi}\right)^2 + \left(\beta\_S_{d(demand)LSi}\right)^2}} \right| \tag{5}$$

$$P_f\left(LSi\right) = \Phi\left(Z\right) \tag{6}$$

For all the ground motion demands scenarios considered in this study, the probabilities of exceeding each limit state are derived for all the four building classes based on the above described methodology. Further, the probabilities of being in a given damage state (DSi) can be arrived as the difference between cumulative probabilities of exceeding consecutive limit states (i.e. DSi = LSi – LSi-1). As an example, Fig.1 shows cumulative probabilities of exceeding the limit states and probabilities of being in specific damage states of post 1976-on building class for increasing shaking demands at 500, 1000, and 2500 year return period derived from NZS 1170.5:2004 design spectra.

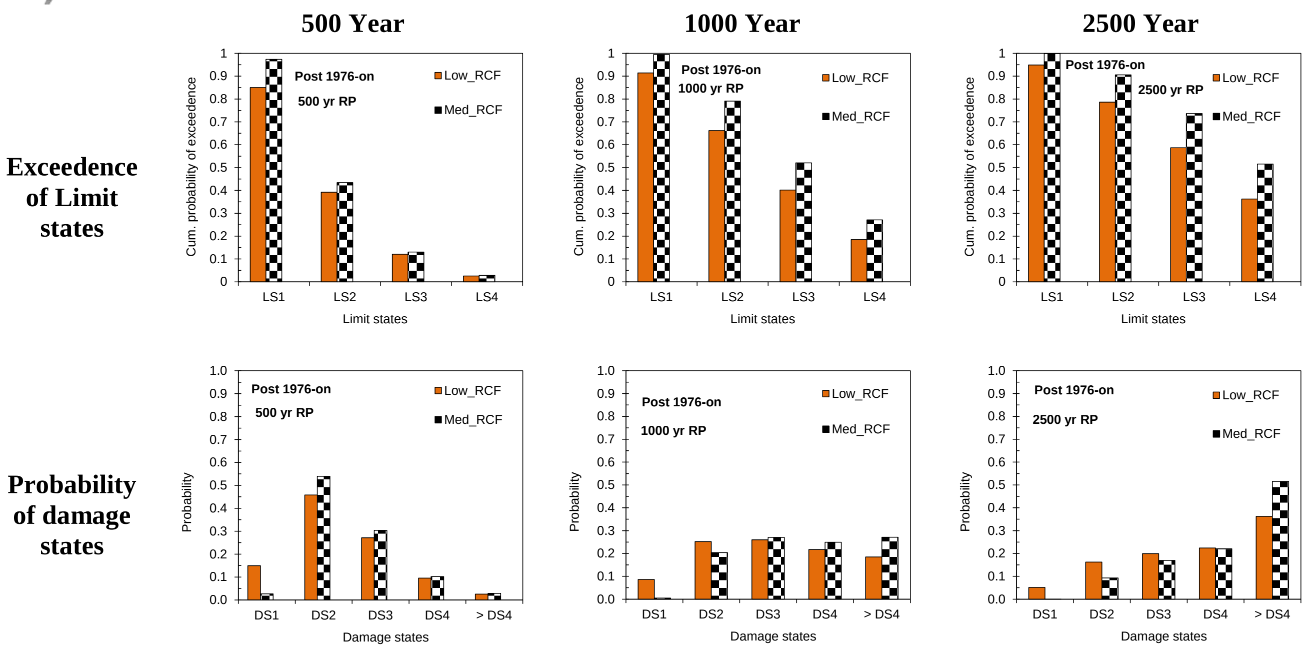

Fig. 1 – Probabilities of damage distributions of low-rise post 1970-on concrete moment resisting frame buildings for varying shaking intensities

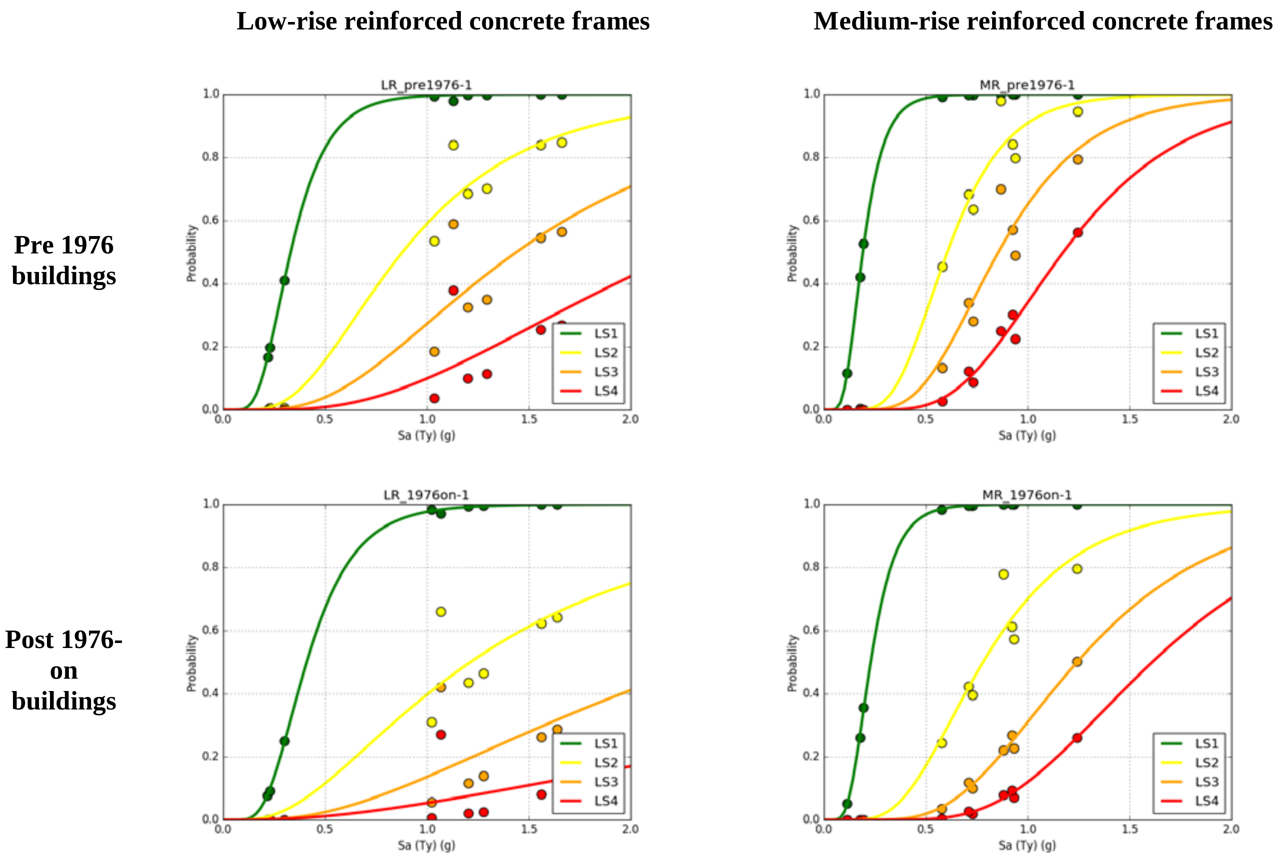

Fig. 2 – Fragility curves developed for low and medium rise reinforced concrete buildings

The limit state probabilities of exceedence obtained for every ground motion scenario are plotted corresponding to the spectral acceleration at the initial period, $S_a(T_y)$ of the building class and lognormal distribution functions are fitted using least squares regression method. Fig.2 shows sets of fragility curves developed for low and medium rise reinforced concrete buildings. Table 3 lists the logarithmic mean (median) and logarithmic standard deviation (dispersion) for all the fragility curves.

Table 3 – Median and dispersion of lognormal fragility functions at each limit states for four building categories

Building Category	Elastic period, s	LS1		LS2		LS3		LS4	
		μ	β	μ	β	μ	β	μ	β
Low rise post 1976	0.5	-0.846	0.816	0.291	0.921	1.084	1.037	2.149	1.386
Low rise pre 1976	0.5	-1.01	0.89	-0.019	0.824	0.581	0.923	1.164	0.951
Med rise post 1976	1.12	-1.456	0.571	-0.244	0.493	-0.218	0.444	0.454	0.39
Med rise pre 1976	1.12	-1.665	0.553	-0.501	0.433	-0.128	0.45	0.201	0.425

6. Damage distribution and color tagging of building stock

The estimates of probabilities of being in a given damage state can as well be interpreted as the proportion of the total buildings (in the group under consideration) falling in that damage state. The damage states are essentially indicators of building safety for occupancy and a calibrated mapping scheme would help to estimate the proportions of buildings in three color categories. In the previous study [10], a mapping scheme between the color tagging and damage states was developed and calibrated with observed damage statistics for RC frame buildings built before and after 1976. The purpose of the mapping scheme is to assist in planning emergency response after an earthquake and prioritising the needs in different areas of a city. As shown in Fig.3, the ‘Green’ tagging includes the whole of DS1 and some portion of DS2; the rest of DS2 combined with DS3 and DS4 are grouped into ‘Yellow’; and anything beyond DS4 is interpreted as ‘Red’. The mapping scheme considered two parameters (α, β) that defined the boundaries of damage states and the parameters were calibrated by minimizing the error between observed and predicted building damage estimates based only on the Christchurch earthquake scenario demand. The values reported in the previous study [10] are reproduced in Table 4.

In the present study, for each building class, the fragility curves are fitted to the probabilities of exceedence of all limit states from various ground motion scenarios, including the Christchurch earthquake scenario. Note that the new fragility curves give fresh sets of damage probabilities estimates at the intensity corresponding to the Christchurch earthquake. Hence, in this study, the mapping scheme has been calibrated once again with the new sets of probabilities of damage states obtained from the fragility curves at the intensities corresponding to the Christchurch scenario with the observed color tagging statistics for all the four building classes. The parameters (α, β) essentially distribute proportions of buildings in different damage states into three color groups. The calibration is done in such a way that the total error between the estimated proportions of tagging and observed proportions of tagging is minimized. Table 5 gives the calibrated values of (α, β) and the corresponding error involved as a result of using the revised mapping scheme. Fig.4 shows the error contours for all the building classes considered in this study.

Table 5 shows that the new sets of (α, β) parameters, based on the newly developed fragility curves in this study, predict less error compared to those suggested in the previous study [10], particularly for low-rise pre 1976 buildings. With the availability of more data the parameters will be further refined. In the meantime, we believe that the proposed mapping scheme and the parameters can be used with reasonable confidence for estimating proportions of buildings with ‘Green’, ‘Yellow’ and ‘Red’ tagging at any given shaking levels of intensity.

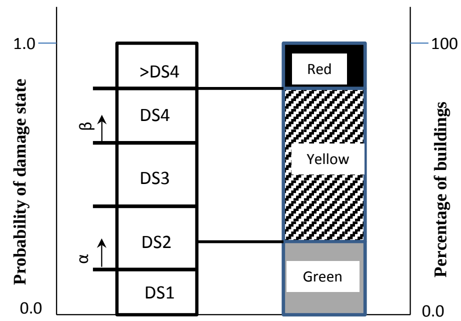

Fig. 3 – Mapping of damage states to tagging colors [10]

Table 4 – Comparison of estimates from mapping scheme with (α, β) with observed color statistics [10]

Category	α (%)	β (%)	Green (%)		Yellow (%)		Red (%)		Error (%)
			Est.	Obs.	Est.	Obs.	Est.	Obs.	
LR Post 1976-On	63	100	23	24	50	53	27	23	5
LR Pre 1976	0	100	3	9	61	72	36	19	21
MR Post 1976-On	45	100	9	11	83	84	8	5	4
MR Pre 1976	0 to 100	100	0 to 2	5	75 to 73	76	25	19	7

Table 5– α and β that minimise the error between the estimates and observed data on color tagging for the Christchurch February 2011 earthquake

Category	α (%)	β (%)	Green (%)		Yellow (%)		Red (%)		Error (%)
			Est.	Obs.	Est.	Obs.	Est.	Obs.	
LR Post 1976-On	48	0	27	24	56	53	17	23	8
LR Pre 1976	26	69	9	9	72	72	19	19	1
MR Post 1976-On	25	100	10	11	83	84	7	5	2
MR Pre 1976	18	100	3	5	74	76	23	19	5

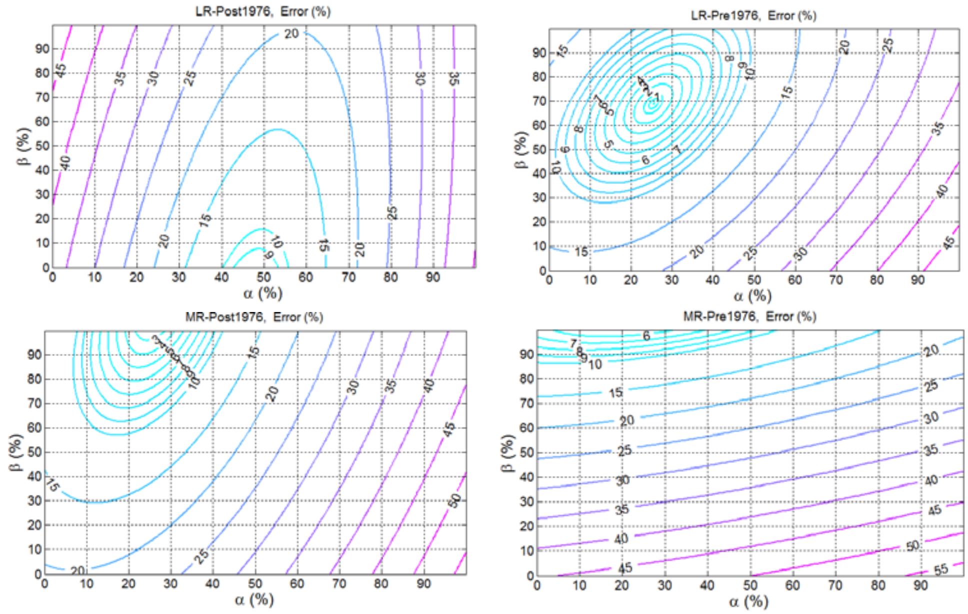

Fig. 4 – Error contours for combinations of proportions of DS2 (α) and DS4 (β) for 'Green' and 'Yellow' bands respectively. Low rise (LR); Medium rise (MR)

The Cook Strait event generated about 0.2g of peak ground acceleration within Wellington City and a spectral acceleration of about 0.25g at the initial period of building classes (0.5s and 1.1s). There was not 'building safety evaluation' exercise was carried out. A general report about the impacts on Wellington buildings can be seen in NZSEE website (http://www.nzsee.org.nz/projects/past-earthquakes/2013-cook-strait-earthquake-sequence/). It is reported that only some medium rise buildings (6-12) storey were significantly affected and most of the low-rise buildings were unaffected. Most damage was related to the non-structural elements (interior fittings), building services, and stairs and ramps. Anecdotally, no buildings were 'Red' tagged that required demolition and many buildings were inspected before re-occupation and a down-time period of 1 to 3 days was observed. Some specific buildings which are located on reclaim land experienced significant damage.

The damage probabilities from the fragility curves at an intensity corresponding to the Cook Strait event were extracted and the refined mapping scheme is used to predict the likely number of buildings in different color tagging. The estimated proportions are listed in Table 6. The numbers reasonably match with the description of damage to buildings in Wellington.

Table 6 – Probable damage states and likely color tagging of building classes under Cook Strait event

Category	**Damage state probabilities**					**Likely color tagging**		
	DS1	**DS2**	**DS3**	**DS4**	**>Ds4**	**Green**	**Yellow**	**Red**
LR Post 1976-On	73	23	3	1	0	81	19	0
LR Pre 1976	65	30	4	1	0	65	35	0
MR Post 1976-On	42	56	2	0	0	55	45	0
MR Pre 1976	29	69	2	0	0	41	60	0

7. Conclusions

New fragility curves are developed for New Zealand reinforced concrete moment resisting frame buildings constructed before and after 1976 based on analytical method. A displacement-based method with a probabilistic framework is adopted to arrive at the building damage distribution.

In this study, ground motion spectral demands were considered in terms of smooth spectra derived based on PSHA. PSHA that is carried out for a specific region or a site considers all possible ground motion scenarios representative of that region and account for aleotory uncertainty (i.e. record to record variability) in a more robust manner. The methodology proposed in this study is simple and practical by not demanding huge numerical and computational efforts in deriving fragility curves. On the other hand, considering a suite of accelerograms requires knowledge and effort in making right choices that could successfully represent the seismicity of that region and yet involves huge computational effort. In addition, the current study has considered real recorded ground motions from Christchurch and Wellington cities to help in constraining the analytical fragility curves and verify the estimated probabilities of failure with the observed damage to buildings in the respective cities.

The Monte Carlo procedure adopted in this study has generated population of buildings that could represent building stock within Wellington (high seismic) and Christchurch (moderate seismic) regions. The spectral demands from high seismic region, i.e. Wellington compared to that from Christchurch are used to give conservative estimates of probability of failures in deriving fragility curves.

In this study damage probability estimates for medium and low rise RC frame buildings given by the new fragility curves are translated into likely color tagging groups using the proposed mapping scheme. For a severe shaking event such as the Christchurch February event, the predicted percentages of buildings in terms of tagging colour (mapped based on their damage states) show close agreement with the observed tagging data obtained from the Christchurch City Council. Further, the damage probabilities from fragility curves at the low intensity shaking level such as the Cook Strait event were similarly translated into likely color tagging groups and compared with the general report on the impact on Wellington buildings during that event. The close correlation between the observed damage and predicted damage probabilities has proved the ability of the fragility curves to provide reasonable estimates of damage probabilities in varying intensities. The results generated from this study are therefore quite appropriate for regional loss estimation and/or emergency planning purposes.

8. Acknowledgements

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9. References

[1] Lin SL, Uma SR, Nayyerloo M, Buxton R, King AB (2014): Characterisation of building performance with detailed engineering evaluation (DEE) data from the Canterbury earthquakes. ASEC 2014 Conference, Structural Engineering in Australasia – World Standards, July 9-11, Auckland, New Zealand.

[2] Rossetto T, Ioannou I, Grant DN (2013): Existing Empirical Vulnerability and Fragility Functions: Compendium and Guide for Selection, GEM Foundation, Pavia, Italy.

[3] Martinelli A, Cifani G, Cialone G, Corazza L, Petracca A, Petrucci G (2008): Building vulnerability assessment and damage scenarios in Celano (Italy) using a quick survey data-based methodology. Soil Dynamics and Earthquake Engineering, 28, 875-889.

[4] ATC (1985): ATC-13, Earthquake damage evaluation data for California. Applied Technology Council, Redwood City, California.

[5] ATC (1996): ATC-40, Seismic evaluation and retrofit of concrete buildings. Applied Technology Council, Redwood City, California.

[6] Borzi B, Pinho R, Crowley H (2008): Simplified pushover-based vulnerability analysis for large-scale assessment of RC buildings. Engineering Structures, 30, 804-820.

[7] Li Y, Ellingwood BR (2007): Reliability of woodframe residential construction subjected to earthquakes. Structural Safety, 29, 294-307.

[8] Rossetto T, Elnashai A (2005): A new analytical procedure for the derivation of displacement-based vulnerability curves for populations of RC structures. Engineering Structures, 27, 397-409.

[9] Crowley H, Pinho R, Bommer JJ (2004): A probabilistic displacement-based vulnerability assessment procedure for earthquake loss estimation. Bulletin of Earthquake Engineering, 2, 173-219.

[10] Uma SR, Dhakal RP, Nayyerloo M (2014): Evaluation of displacement-based vulnerability assessment methodology using observed damage data from Christchurch. *Earthquake Engineering & Structural Dynamics*, **43**, 2319-2339.

[11] Silva V, Varum H, Crowley H, Pinho R (2012): Extending displacement-based earthquake loss assessment (DBELA) for the computation of fragility curves. 12th World Conference on Earthquake Engineering, LISBOA, Portugal.

[12] Crowley H, Bommer JJ, Pinho R, Bird J (2005): The impact of epistemic uncertainty on an earthquake loss model. *Earthquake Engineering & Structural Dynamics*, **34**, 1653-1685.

[13] Bal IE, Crowley H, Pinho R, and Gulay FG (2008): Detailed assessment of structural characteristics of Turkish RC building stock for loss assessment model. Soil Dynamics and Earthquake Engineering. 28, 914-932

[14] Rosetto T, Elnashai A (2003): Derivation of vulnerability functions for European-type RC structures based on observational data. Engineering Structures, 25, 1241-1263.

[15] Pinho R, Bommer JJ, Glaister S (2002): A simplified approach to displacement-based earthquake loss estimation analysis. Proc. of the 12th European Conference on Earthquake Engineering, England, no. 738.

[16] Glaister S, Pinho R (2003): Development of a simplified deformation-based method for seismic vulnerability assessment. Journal of Earthquake Engineering, 7, Special issue, 107-140.

[17] NZS1170.5:2004 (2004): Structural Design Actions Part 5 Earthquake actions - New Zealand, Standards New Zealand, Wellington, New Zealand.

[18] Priestley MJN, Calvi, GM and Kowalsky, MJ (2007): Displacement-based seismic design of structures. IUSS Press, Pavia, Italy.