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Comparison of main-shock and aftershock fragility curves developed for New Zealand and US buildings

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ABSTRACT:

Seismic risk assessment involves the development of fragility functions to express the relationship between ground motion intensity and damage potential. In evaluating the risk associated with the building inventory in a region, it is essential to capture 'actual' characteristics of the buildings and group them so that 'generic building types' can be generated for further analysis of their damage potential. Variations in building characteristics across regions/countries largely influence the resulting fragility functions, such that building models are unsuitable to be adopted for risk assessment in any other region where a different set of building is present. In this paper, for a given building type (represented in terms of height and structural system), typical New Zealand and US building models are considered to illustrate the differences in structural model parameters and their effects on resulting fragility functions for a set of main-shocks and aftershocks. From this study, the general conclusion is that the methodology and assumptions used to derive basic capacity curve parameters have a considerable influence on fragility curves.

1 INTRODUCTION

Regional seismic risk assessment requires building fragility functions to be developed for building portfolios to represent probabilities of potential damage due to earthquake hazard. The regional building portfolio is divided into various building classes based on structural system, height and construction material. Further, a typical building is identified with certain parameters to represent that building class. Note that the generic characteristics of typical buildings vary across countries and, therefore, building fragility functions developed for one region may not be appropriate to be used in some other region where a different building portfolio is to be represented. For example, HAZUS (1999), a risk assessment tool, uses fragility functions specifically for the US building inventory. The parameters used for developing these fragility functions were mostly based on expert opinion and engineering judgement. To assess seismic risk in any other country with different building characteristics, the HAZUS based fragility functions may not be suitable. In New Zealand, 'Riskscape' a multi-hazard risk assessment tool (under joint development by GNS and NIWA), includes a building classification system similar to HAZUS, but for typical NZ buildings (King et al., 2009).

Risk assessment tools often consider risk due to a main-shockmain-shock event only. However, it is not uncommon to get many aftershocks after the main-shockmain-shock, some of which could be strong enough to cause further damage to the building. In such situations, it is necessary to estimate the residual capacity of main-shock damaged buildings. Luco et al., (2004) have addressed a methodology to determine the residual capacity of main-shock damaged buildings which could be adopted to develop 'aftershock fragilities'. A methodology to derive these aftershock fragility curves has been proposed by Ryu et al. (2011).

Considering the variability in building characteristics between regional building classes, an attempt has been made by the authors to illustrate the differences in fragility functions between US and NZ building models. In this regard, a typical five storey building representing medium-rise reinforced concrete moment resisting frames has been chosen to be modelled to represent US and NZ building stock, respectively.

Non-linear incremental dynamic analyses are carried out on both US and NZ models for a suite of main-shock and aftershock records. Five different damage states are defined, slight, moderate, extensive, complete and collapse, and associated fragility functions developed. Damage state thresholds are defined based on criteria established in previous work (Ryu, 2008). Fragility functions are derived for US and NZ models for a main-shock and possible aftershocks event of magnitude (M) within a specified range.

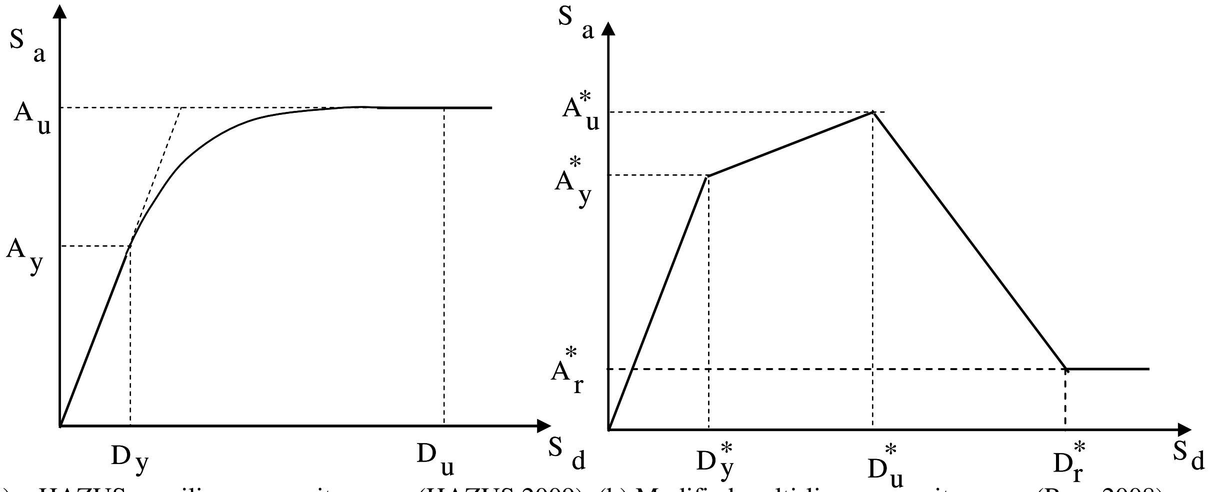
2 GENERIC BUILDING MODELS FOR REGIONAL RISK ASSESSMENT

One of the biggest challenges in deriving a fragility/vulnerability model is to acquire an appropriate building inventory database. In developing building classification systems for Riskscape, pilot studies were conducted on three regions (Christchurch, Hawke's Bay and Westport) to represent the building types common within New Zealand. The three regions were chosen as representatives of distinctly different categories, viz. large city (about 300,000 buildings), small city/rural (30,000 buildings) and town (2000 buildings). Generic buildings are defined based on the building characteristics, including height and structural system adopted to resist lateral loads. HAZUS has included a total of 36 generic building models for the US building inventory. In Riskscape, 18 building classes have been identified for the NZ building inventory and generic characteristics of building classes are listed elsewhere (King et al, 2009). It is worth mentioning that while developing building classifications for Riskscape, it was kept in mind to follow similar grouping systems with respect to number of storeys as in HAZUS; i.e. (a) low-rise buildings (up to 3 storeys); (b) medium-rise buildings (4-7 storeys); and (c) high-rise (8 storeys or more) in the interest of seeking some common basis.

It is to be noted that the fragility functions for 'generic buildings' are developed based on the response of a typical building with generic structural properties and hence cannot be directly applicable for building-specific risk assessment purposes. The structural models for a generic building within a building class should preferably be determined after accounting for the variability in building characteristics of that building class.

2.1 Parameters for capacity curves

Estimation of building response requires developing representative building models either in the form of single-degree-of-freedom (SDOF) models or multi-degree-of-freedom (MDOF) models. Unlike a MDOF model where building details are explicitly specified, a SDOF model is defined using a capacity curve with a certain number of control points. For example, the HAZUS methodology proposed curvilinear capacity curves as shown in Figure 1 (a) using two sets of control points at yield (A_y, D_y) and ultimate (A_u, D_u) capacities. The HAZUS capacity curve remains plastic, without any strength degradation after reaching ultimate capacity, which is unrealistic. Also, the ratio of ultimate to yield displacement, (i.e., the effective ductility), is too large for real structures. This is because the ultimate displacement capacity is not the 'true' ultimate displacement capacity of the system. It is just a point along the capacity curve at which the maximum strength has been fully attained. SDOF models based on these parameters are appropriate for use in the capacity spectrum method, and not where non-linear time history analyses are involved.



(a) HAZUS curvilinear capacity curve (HAZUS 2009) (b) Modified multi-linear capacity curve (Ryu, 2008)

Figure 1 Capacity curve definitions for single-degree-of-freedom (SDOF) model

As an alternative to the HAZUS curvilinear curve, Ryu et al (2008) proposed a multi-linear capacity curve with a negative stiffness after the ultimate (capping) point to include degradation in system performance. The proposed multi-linear capacity curve has yield (A_y*, D_y*), ultimate (A_u*, D_u*) and residual (A_r*, D_r*) capacity points, which are more suitable to describe non-linear dynamic SDOF models.

In this study, building models for a five storey reinforced concrete moment-resisting frame are idealised as SDOF models and defined with multi-linear capacity curves. The basic parameters necessary to define the model are the displacement and base shear coefficient at the yield point (A_y*, D_y*) where 'significant yield' is expected; the ultimate displacement, defined by structural ductility; and the ultimate capacity, defined in terms of the strain-hardening ratio with respect to yield capacity. The residual strength is assumed as 20% of yield strength. The residual displacement (D_r*) is considered to be coinciding with 'collapse' damage state thresholds and the values are given in Table 2.

The bases for selecting parametric values on the multi-linear curves for NZ and US models are discussed below.

2.2 NZ building model

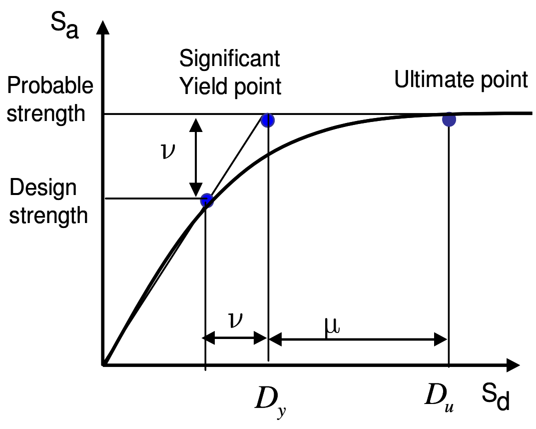


Figure 2. Idealised capacity curve (Ref: Park, 1997)

A typical five storey reinforced concrete frame with a total height of 18m is considered for this study. The proposed capacity curve is given in Figure 2 (Park, 1997). The 'design strength' refers to the code-specified lateral strength where the first plastic hinge is assumed to be forming. Further plastic hinges form to reach the 'significant yield point' where a mechanism forms. The probable strength is obtained using a factor, ν, to account for the probable overstrength of the material (taken as 1.25) and the redundancies (taken as 1.75) in the structural system. The 'ultimate point' is ductility 'μ' times the yield displacement. Based on a displacement based approach (Priestley et al., 2007), the yield displacement is determined. This approach uses mechanically-derived formula (or equations) to describe yield displacement capacity using geometrical and material properties. A Monte-Carlo procedure is adopted to simulate the geometrical and material property variables for the typical building. The structural characteristics of the NZ building model are assumed to be within the range of values assigned for medium-rise buildings as shown in Table 1. Note that U[ ] represents uniform distribution and N[ ] represents normal distribution for the variables. Further details on the range of variables considered for simulation for a medium-rise reinforced concrete moment-resisting frame are presented elsewhere (Uma et al, 2010). From simulation, the median displacement is chosen as the yield displacement for the model. A limited ductility of 3 is considered so that ultimate point is close to a 2% drift ratio; the ultimate strength at ultimate point is obtained with a low post-yield stiffness ratio of 5%.

The initial period is computed based on the recommendations in the Commentary to NZS1170.5:2004 (SNZ, 2004). The building periods, based on code recommendations, are usually conservative for estimating design base shear, and less than the 'true' value. The median initial period is estimated on the higher side, considering a reasonable amount of variation from the initial period recommended for design purposes. The initial period for the NZ model is taken as 1.3s. The design strength is obtained from NZ 1170.5:2004 design spectra for site subsoil class 'C' and for the Wellington region with a hazard coefficient of Z =0.4, and the probable strength is obtained after accounting for overstrength factors as mentioned above.

Table 1. Structural parameters for concrete moment-resisting frame structures.

Structural Parameters	Range of values	Structural Parameters	Range of values
Number of storeys, N_s	U [4,7]	Beam depth (m), h_b	U [0.5, 0.7]
Storey height (m), S_h	U [3.4, 3.8]	Steel strength (MPa), f_y	N [325,35]
Beam length (m), l_b	U [5.0, 7.0]	Effective height coeff., *efh*	0.64-0.0125*(N_s-4)

2.3 US building model

A comparable HAZUS building type C1M with a HAZUS-suggested ductility of 5.3 is chosen to represent a typical five storey building with a height of 50 feet (about 15.24 m). The original HAZUS-based capacity curve parameters are notably unrealistic. In this regard, Ryu et al., (2008) suggested a modified procedure to construct a multi-linear capacity curve where the yield and ultimate capacity points are determined via an iterative procedure.

Table 2. Parameters to define multi-linear capacity curves for NZ and US models.

	Ty, s	Yield		Ultimate		Residual	
		D_y, m	A_y*, g	D_u, m	A_u*,g	D_r, m	A_r*, g
NZ model	1.3	0.08	0.20	0.24	0.22	0.44	0.044
US model	0.75	0.06	0.46	0.34	0.62	0.61	0.09

The building period is taken as 0.75s as suggested by HAZUS. The original yield strength, A_y, accounts for overstrength and is about 0.2g which is very close to the probable strength of the NZ model. The iterative procedure is based on 'equal area principle' within the curvilinear portion and assumes the initial stiffness suggested by HAZUS which is unaltered for determining the 'significant yield' point (Ryu, e al., 2008). The yield base shear coefficient A_y obtained from the above iterative procedure resulted in a much higher value than that for the NZ building. The ultimate displacement point is ductility times the significant yield point. The ultimate capacity is taken with an 8.5% strain-hardening ratio from yield capacity, and the residual capacity is 20% of the ultimate capacity. Figure 3 shows the plots of multi-linear capacity curves for NZ and US building models and Table 2 lists the values.

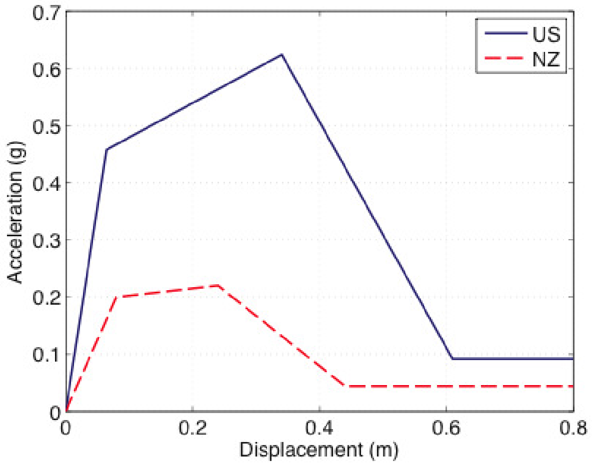


Figure 3. Multi-linear capacity curves for NZ and US models

3 GROUND MOTIONS

The suite of thirty records compiled by Vamvatsikos and Cornell (2006) is used for both main-shock and aftershock records. The moment magnitude of the records is within 6.5-6.9, and the closest distance to fault rupture of the records is within 15-33km. The fundamental period of the US model is 0.75sec and that for the NZ model is 1.3 s. Spectral acceleration at 1.3 s with a damping ratio of 5% is chosen as the ground motion intensity measure for both of the models. Selection of the Sa(T=1.3s) intensity measure is mainly for comparison of the fragility curves generated by the models; it is justified to choose a longer period than the fundamental period because 1) the system will have a longer period if it becomes inelastic or nonlinear; 2) in incremental dynamic analyses (IDA) curves, responses from longer periods show less variability.

4 FRAGILITY CURVES

Fragility curves are expressed as cumulative lognormal distribution curves and are developed for five damage states. The median damage state threshold values in terms of roof displacement are given in Table 3 for the NZ and US building models.

Table 3. Damage state thresholds considered for NZ and US models

Damage state	Description	NZ model (m)	US model (m)
1	slight	0.08 (0.7%)	0.06 (0.5%)
2	moderate	0.16 (0.14%)	0.16 (0.14%)
3	extensive	0.24 (2.0%)	0.34 (3.0%)
4	complete	0.29 (2.6%)	0.40 (3.5%)
5	collapse	0.44 (3.9%)	0.61 (5.3%)

The methodology to derive fragility curves considering the uncertainty in damage state thresholds is discussed in a companion paper (Hyeuk, 2011). A lognormal standard deviation of 0.4 is considered to represent the uncertainty in damage state thresholds.

4.1 Fragility curves for mainshocks

Incremental dynamic analyses are performed on SDOF non-linear models described by multi-linear capacity curves with parameters as shown in Figure 2. The time history analyses adopt a pinching hysteretic model to simulate strength and stiffness degradation within the system. The procedure to develop fragility curves from incremental dynamic analyses is described in detail in Ryu et al. (2011). The fragility curves derived for US and NZ models are shown in Figure 4. It is clear that the median Sa(T=1.3s) values for the US models are higher than that those for the NZ models. The reason is that the US model is characterised by higher capacity and is associated with damage state threshold points at larger drift ratios.

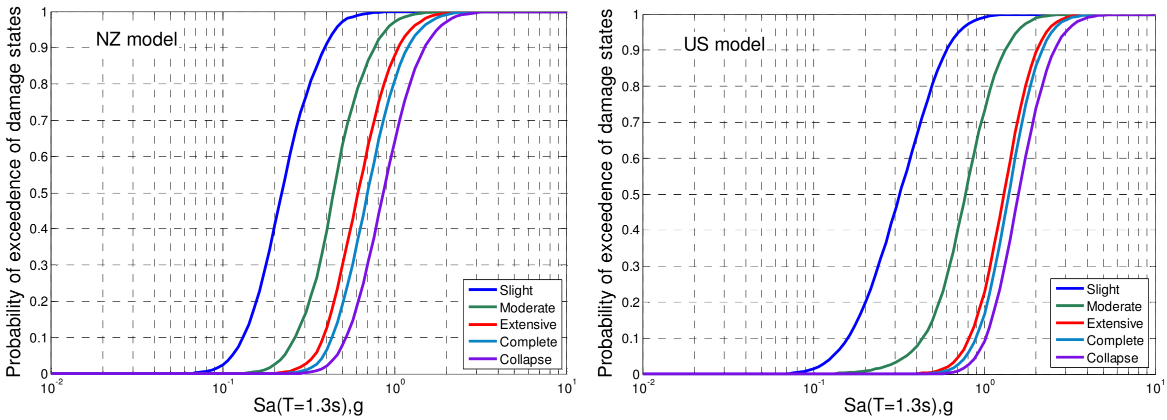


Figure 4. Fragility curves for all damage states due to main-shock records on NZ and US models

In order to compare the fragility curves from NZ and US building models, a common basis is established by setting the damage threshold points for the US model the same as those for the NZ model. A set of comparison plots for four damage states for the 'modified' US model and NZ model is shown in Figure 5.

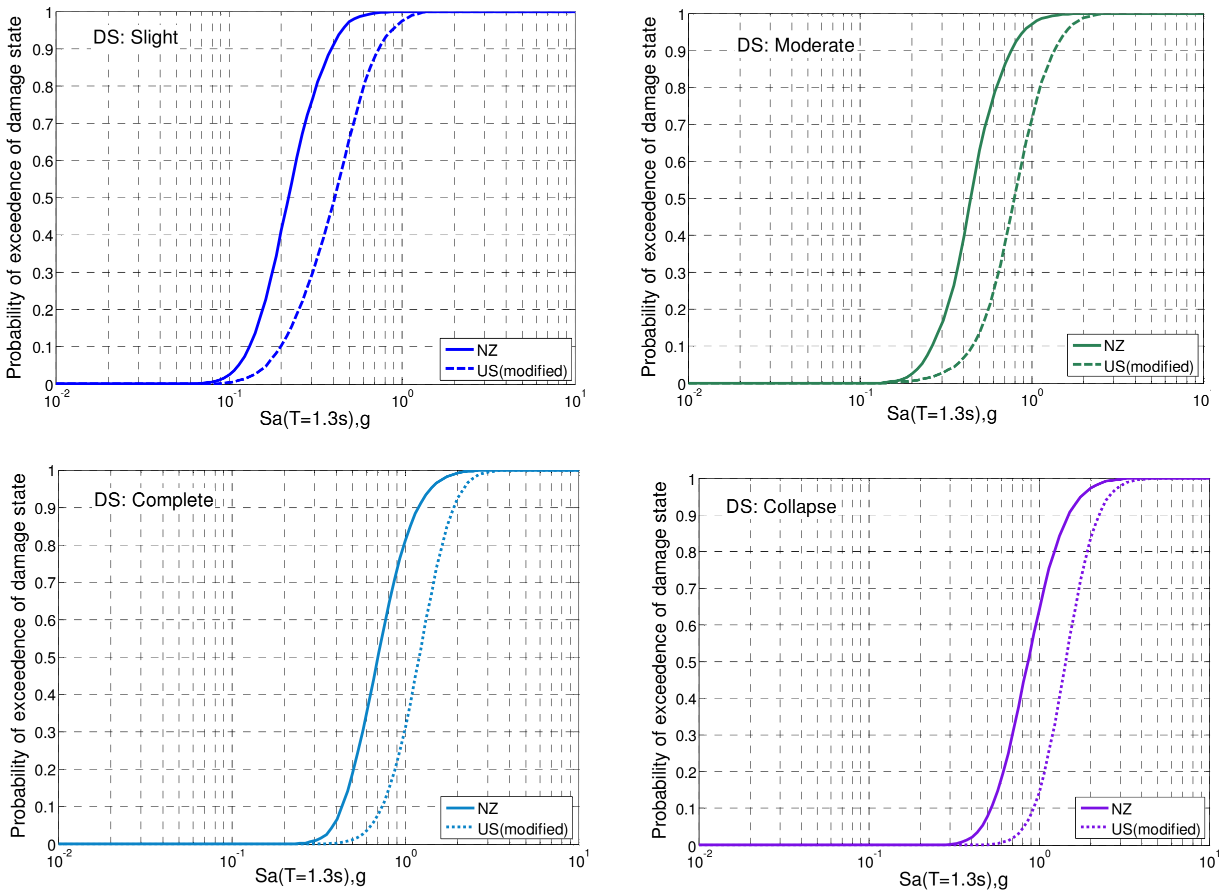


Figure 5 Comparison of fragility curves for the NZ and US models for damage states: Slight, Moderate, Complete and Collapse

Table 4. Median values of Sa (*T*=1.3s)g and damage threshold drift ratios for various damage states

	Slight		Moderate		Extensive		Complete		Collapse	
	Sa,g	Dr(%)	Sa,g	Dr(%)	Sa,g	Dr(%)	Sa,g	Dr(%)	Sa,g	Dr(%)
NZ	0.22	0.7	0.44	1.4	0.61	2.0	0.70	2.6	0.87	3.9
US(modified)	0.41	0.7	0.80	1.4	1.12	2.0	1.20	2.6	1.45	3.9
US	0.32	0.5	0.78	1.4	1.32	3.0	1.39	3.5	1.58	5.3

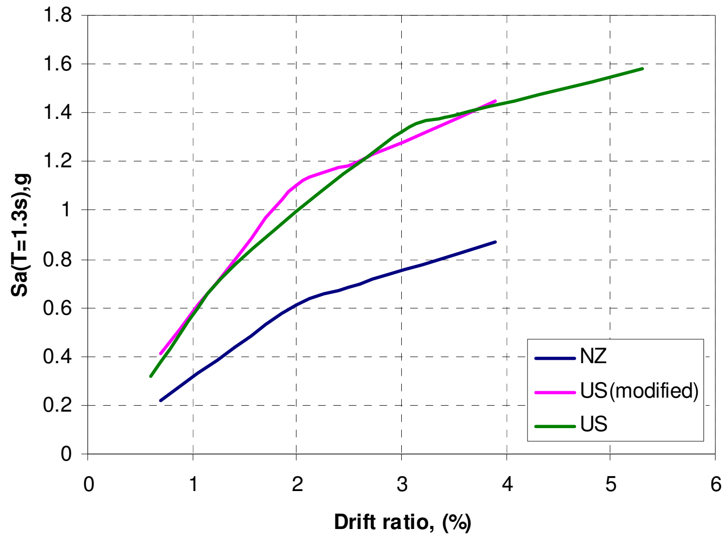


Figure 6 Variation of Sa(T=1.3s),g with respect to drift ratio for NZ model and US models with different sets of damage threshold

From the fragility curves shown in Figures 4 and 5, only the median Sa values at all damage states are plotted against drift ratios in Figure 6. It is evident that Sa values are influenced by the stiffness and strength of the building models. The Sa values of the US models are about 1.9 times those for the NZ model up to the 'Extensive' damage state and about 1.7 times those for NZ model for 'Complete' and 'Collapse' damage states. Overall, it is apparent that the fragility functions are highly sensitive to the maximum capacity of the building model.

4.2 **Fragility curves for aftershocks**

In addition to comparison of main-shock fragilities between US and NZ models, we compared fragilities for buildings damaged under the main-shock. For this illustration, post-main-shock damage is assumed to be in the extensive damage state, and the post-main-shock response is assumed to follow a lognormal distribution, with a median damage state threshold for the 'Extensive' damage state of 0.24 and 0.34 for the US and NZ models respectively. For each realisation of the main-shock-damaged model, which was simulated by subjecting the model to a main-shock record to get it to have the predefined post-main-shock response, we perform incremental dynamic analyses using the aftershock records. For each main-shock record (specifically, a realization of main-shock damage due to a particular main-shock), 30 aftershocks are applied to estimate seismic demands on main-shock-damaged building. The procedure to compute fragility for aftershocks is described in detail in Ryu et al. (2011). Figure 7 compares the collapse fragilities when the models are in an 'Extensive' damage state due to the main-shock. It is clear that the residual

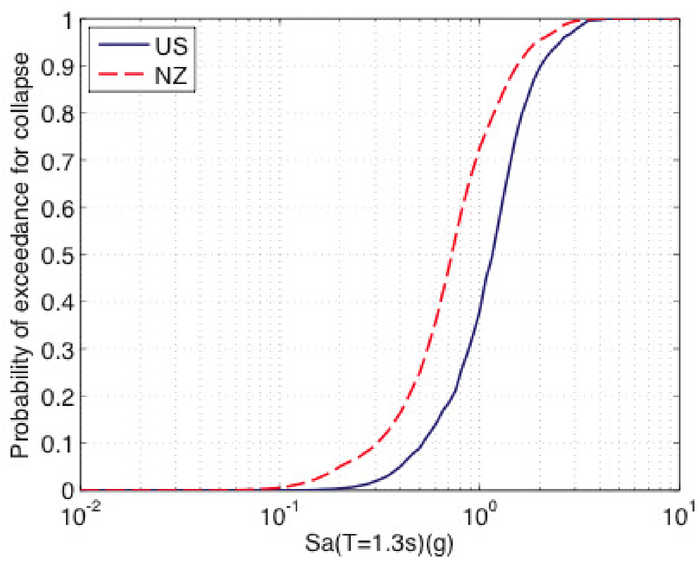


Figure 7. Aftershock fragility curves for NZ and US models

capacities of the buildings having 'Extensive' damage from 'Collapse' (1.16g and 0.73g for US and NZ buildings respectively: Figure 7) are less than those for intact (undamaged) buildings to from reaching the Collapse damage state (1.59g and 0.86g for US and NZ buildings respectively: Figure 4).

5 SUMMARY

In this study, fragility functions developed for five storey buildings typically representing medium-rise reinforced concrete frames in US and NZ are presented. The dynamic responses of the buildings are determined by incremental dynamic analyses of SDOF models. The differences in fragility functions between the US and NZ models arise because of the assumptions involved in developing the capacity curve parameters for the SDOF models. Some level of engineering judgement and empirical expressions are used to arrive at the control parameters. The variability in building characteristics to represent a building class is considered through simulation. In general, modified HAZUS parameters for both drift and strength that define the US model are higher than the parameters evaluated for the NZ model.

Since the fragility functions are influenced by the basic capacity curve parameters and the procedure involves considerable computational effort in carrying out incremental dynamic analyses (IDA), it is an imperative that the parameters for SDOF models are predicted with better approaches (e.g. by pushover analyses on MDOF models) and not only based on engineering judgement. Currently, work is ongoing in developing fragility curves for older reinforced concrete frames modelled as two dimensional frames with non-ductile beam, column and joint elements.

From the present study, it appears that the NZ models are more fragile than US models, both with regard to main-shocks and aftershocks, but this observation is not conclusive without carrying out detailed studies with better structural models representing 'true' characteristics to predict their non-linear dynamic responses.

6 ACKNOWLEDGEMENTS

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