ECONOMIC PAYBACK OF IMPROVED DETAILING FOR CONCRETE BUILDINGS WITH PRECAST HOLLOW-CORE FLOORS

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SUMMARY

A seismic financial risk analysis of typical New Zealand reinforced concrete buildings constructed with topped precast concrete hollow-core units is performed on the basis of experimental research undertaken at the University of Canterbury over the last five years. An extensive study that examines seismic demands on a variety of multi-storey RC buildings is described and supplemented by the experimental results to determine the inter-storey drift capacities of the buildings. Results of a full-scale precast concrete super-assemblage constructed and tested in the laboratory in two stages are used. The first stage investigates existing construction and demonstrates major shortcomings in construction practice that would lead to very poor seismic performance. The second stage examines the performance of the details provided by Amendment No. 3 to the New Zealand Concrete Design Code NZS 3101:1995. This paper uses a probabilistic financial risk assessment framework to estimate the expected annual loss (EAL) from previously developed fragility curves of RC buildings with precast hollow core floors connected to the frames according to the pre-2004 standard and the two connection details recommended in the 2004 amendment. Risks posed by different level of damage and by earthquakes of different frequencies are examined. The structural performance and financial implications of the three different connection details are compared. The study shows that the improved connection details recommended in the 2004 amendment give a significant economic payback in terms of drastically reduced financial risk, which is also representative of smaller maintenance cost and cheaper insurance premiums.

1. INTRODUCTION

Concrete buildings that use precast prestressed hollow-core floor units have been the dominant form of construction in New Zealand (NZ) over the last three decades. Failures observed after the 1994 Northridge earthquake have raised some concerns regarding the performance of NZ's multi-storey moment resisting RC frame buildings having precast concrete hollow-core floors. This is because NZ construction methods are similar to those used in the US and several US precast buildings did not perform adequately during the Northridge earthquake. During this earthquake several buildings collapsed as a result of the hollow-core flooring units losing their seating from the supporting beams [1]. Once the beam support was lost, the units collapsed onto the floor below.

Based on their experimental investigations Matthews et al. [2] and Lindsay et al. [3] integrated aspects of capacity versus demand by developing a series of probabilistic based fragility curves. These curves are further extended in the present work to include financial loss estimation. An earthquake – recurrence relationship is defined to transform spectral acceleration to annual frequency. A loss ratio, which is the ratio of the repair cost necessary to restore the full functionality of the structure to the replacement cost, is then assigned to each damage state observed experimentally. Expected annual loss is calculated using the extension of the PEER triple integral formulation [4], extended by Dhakal and Mander [5] to a quadruple integral equation. A comparison in the estimated loss of pre- and post- amendment precast concrete buildings of New Zealand is made and discussed. Limitations of the study and sensitivity to various parameters are reported. Comments useful to owners and insurers of the buildings are made from an insurance point of view. Work done by Matthews [6], Lindsay [7] and MacPherson [8] is adopted in the present paper as a basis for this economic analysis.

2. SUMMARY OF PREVIOUS WORK

After observing the failures in Northridge a multi-stage study was undertaken at the University of Canterbury, to determine whether NZ designed and built structures have similar problems, and if so, to what extent these problems exist and what can be done about them.

At first, an extensive study that examined the seismic demands on a variety of precast concrete multi-storey buildings was conducted by Matthews [6]. Experimental studies were then performed in two stages to determine the inter-storey drift capacities of multi-storey RC buildings with precast concrete hollow-core floors. A series of large scale experiments were conducted on a full scale super-assemblage in order to ascertain the inter-storey drift corresponding to various damage states. Stage 1 of the experimental study examined the then-existing precast

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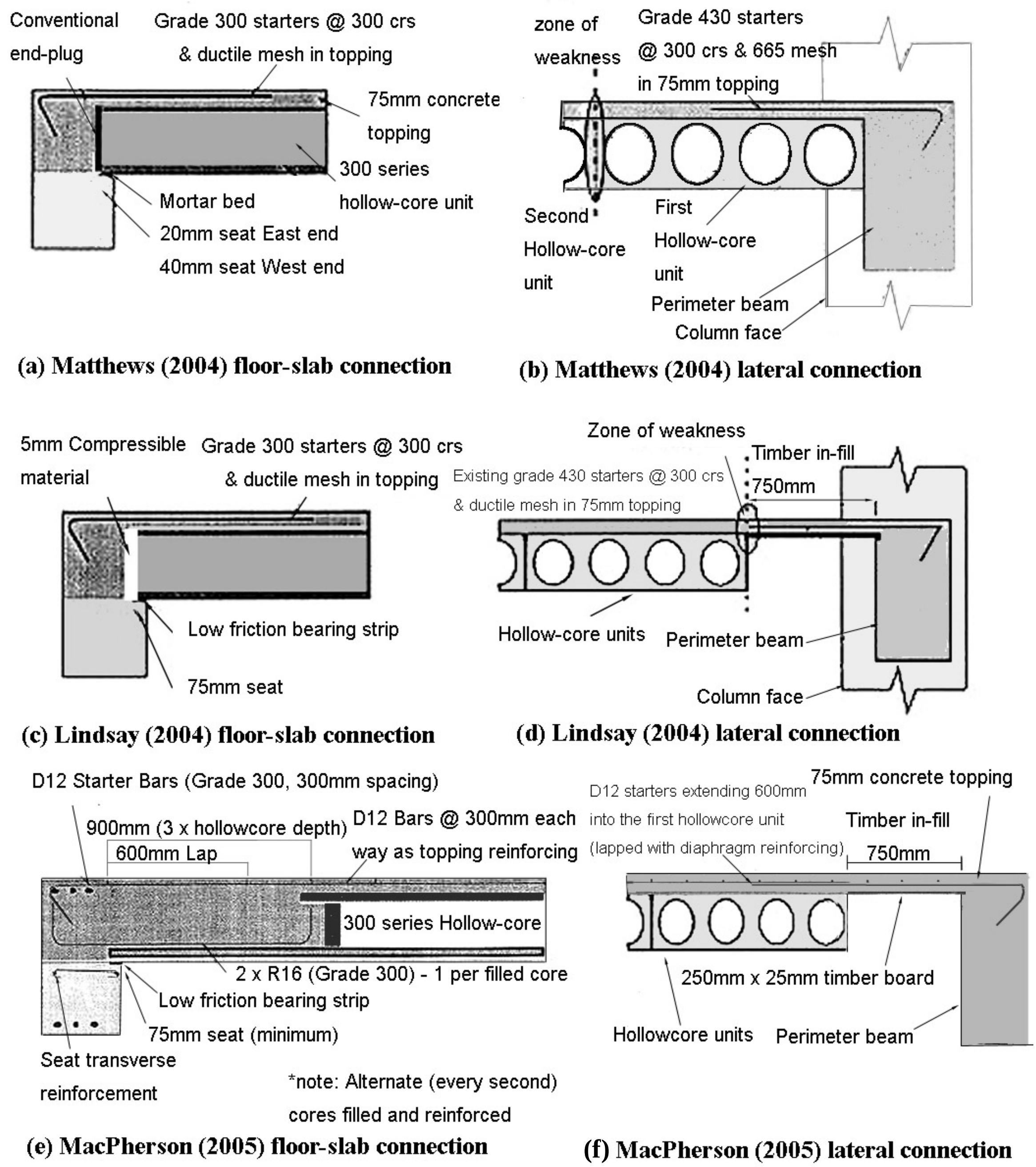


Figure 1: Hollow-core connection details used by Matthews [6], Lindsay [7] and MacPherson [8].

concrete detailing practice in NZ, as recommended by the NZ concrete standard NZS3101:1995 [9]. The collapse of hollow-core units during the tests by Matthews [6] in stage 1 flagged issues over the performance of existing precast concrete frame structures with hollow-core flooring structural systems. In stage 2, Lindsay [7] and MacPherson [8] tested and reported the improved performance of similar super-assemblage incorporating the floor-frame connection details as recommended in Amendment No. 3 to the New Zealand Concrete Design Code NZS3101:1995 [9].

2.1 Experimental Assessment of Drift Capacity

A full scale super-assemblage experimental set-up was conceived and a new testing methodology was developed to investigate the 3D seismic performance of concrete frames with precast floors. The super-assemblage specimen was a two-bay by one-bay section of a lower storey in a multi-storey RC moment resisting frame. The floor units were pre-tensioned prestressed precast hollow-core units that were oriented so that they run parallel with the long edge of the building, past a central column. The connection details of hollow-core units used in the experimental programme are shown in Figure 1. The super assemblage was tested in two stages as follows:

Stage 1: Matthews [6] first tested the super-assemblage specimen, emulating the 1980's and 1990's construction practice that has historically become the norm in NZ. The reinforcing details were in accordance with NZS3101:1995 [9]. Due to inadequate seating (Figure 1a), as well as displacement incompatibilities between the frame and the floor (Figure 1b), the experiment showed that premature failure of the flooring system can be expected for design basis earthquakes in NZ. It was demonstrated that the floor-to-beam seat connections of existing precast concrete construction are particularly vulnerable.

Stage 2: Lindsay [7] repaired the damaged plastic hinge zones in the frame, and then reconstructed the floor by using modified seating details. Amendment No.3 to NZS3101:1995 [10] provides two details for the connection of hollow-core floor units to the supporting beams. Lindsay [7] reported on the performance of the first of these, with particular attention paid to addressing following three aspects of structural detailing:

1. Improving the seating connection detail between the precast, prestressed hollow-core floor diaphragm and the perimeter reinforced concrete moment resisting frame (Figure 1c).
2. Stopping the central column from displacing laterally out of the building due to an insufficient lateral tie into the building. It was because of this lack of interconnection that the floor slab tore longitudinally due to displacement incompatibility in Matthews test. The central column was not restrained and was able to translate freely outwards in Matthews test.
3. Isolating the first hollow-core unit spanning parallel to the perimeter beams from the frame to avoid displacement incompatibility (Figure 1d). This displacement incompatibility was caused by the units being forced to displace in a double curvature manner due to being effectively connected to the edge of the perimeter beam, when hollow-core units are not designed for such displacement profiles.

The second detail of NZS3101:1995 [9] specifies a reinforced connection that rigidly ties the floor into the supporting beam (Figure 1e). MacPherson [8] investigated the effectiveness of this solution by re-testing the large-scale three-dimensional specimen. The super-assemblage tested by MacPherson included the following details:

1. A reinforced connection that rigidly ties the floor into the supporting beam (Figure 1e).
2. An articulated topping slab portion cast on a timber infill that runs parallel to and connects the hollow-core units and edge beams (Figure 1f).
3. Specially detailed supporting beam plastic hinge zones to reduce potential damage to the hollow-core units.
4. Grade 500E reinforcing steel used in the main frame elements.
5. Mild steel deformed bars in the concrete topping in lieu of the customary welded wire mesh.

2.2 Classification of observed building damage

A common form of damage classification is to use a numerical indicator format as adopted by HAZUS [11]. As given in Table 1, numbers from one and five that refer to increasing level of damage are used.

Table 1 HAZUS classification of damage states following an earthquake [11]

Damage State	Damage Descriptor	Post-earthquake Utility of Structure
1	None (pre-yield)	Normal
2	Minor / Slight	Slight Damage
3	Moderate	Repairable Damage
4	Major / Extensive	Irreparable Damage
5	Complete Collapse	

Based on post-earthquake utility and life-safety considerations, the drift limit states for different level of damage are summarised in Table 2. The values of drifts corresponding to different damage states listed in the 2^{nd}, 3^{rd}, and the 4^{th} columns of Table 2 have been decided based on experimental results [6-8]. Similarly, the drifts corresponding to different level of damage of the seismic frames stipulated in the last column of Table 2 have been decided based on the requirements of NZ standards [9, 10].

Table 2 Damage state classification for the super-assemblage

Damage State	Inter-storey drift based on:			
	Historical[@] floor detailing practice	Modern[#] detailing practice for floors and their connections		Historical[@] and current[#] frame detailing practice
		Detailing 1	Detailing 2	
2	0.3%	1%	1%	1%
3	0.35%	2%	2%	2%
4	1.9%	2.25%	4%	4%
5	2.5%	5%	5%	6%

[@] 1985-2003 details to NZS 3101

[#] 2004 Amendment 3 to NZS 3101:1995

Note that the drift values given in Table 2 are the global inter-storey drift which would have caused different level of damage in the specified component (floor, frame) provided that the other components of the building remain perfect. Hence, the inter-storey drifts corresponding to frame damage (final column of Table 2) are immaterial in buildings with precast hollow-core floors as the floor or the floor-to-frame connections (prior to the frame) would damage to a similar or larger extent at the same global inter-storey drift. Nevertheless, these values help realize the extent of weakness the floors with different connection detail impart to the building.

For example, it is apparent that the building would have minor or no damage until 2% drift if the floors were not included or if the floors and the connections were perfect. The inclusion of precast hollow-core floors with pre-2004 connection detail weakens the building to such an extent that it would be severely and irreparably damaged at 2% drift. Despite implementing the improved detailing 1, performance of the building with floor will still be weaker compared to that of building with no floors or perfect floors. As can be seen in Table 2, the building with improved floor-frame connections using detailing 1 would have extensive or irreparable damage at 2.25% drift, whereas at the same level of drift similar buildings with perfect/no floors would experience repairable moderate damage only. Further improvement of the floor-frame connections using detailing 2 would bring the building performance almost on par with the frames; in other words, floors with detailing 2 will not impair the building performance.

2.3 Assessment of Drift Demand

Matthews [6] used the approach developed by Cornell et al [12] for steel structures and further extended by Lupoi et al [13] for the seismic design of reinforced concrete structures for probabilistic assessment of drift demand on a family of seismically vulnerable multi-storey concrete buildings with precast hollow-core floor units designed and constructed in

NZ during the period from 1985 to 2003. To assess the expected seismic demands on a concrete structure, nonlinear time history analyses were undertaken. In order to simulate the likely seismic performance of the test buildings, a suite of earthquake records was chosen for the time history analysis. The dimensions of the "prototype buildings" investigated were based on a representative sample of buildings idealised from professional practice in NZ from the 1980's through 1990's. Results of the time history analyses were normalised so that all the various forms of earthquake motions had a common variable. Results were plotted in the form of cumulative distribution versus drift index proportionality parameter 'a = Drift / Spectral Acceleration (F_vS_1)' as shown in Figure 2.

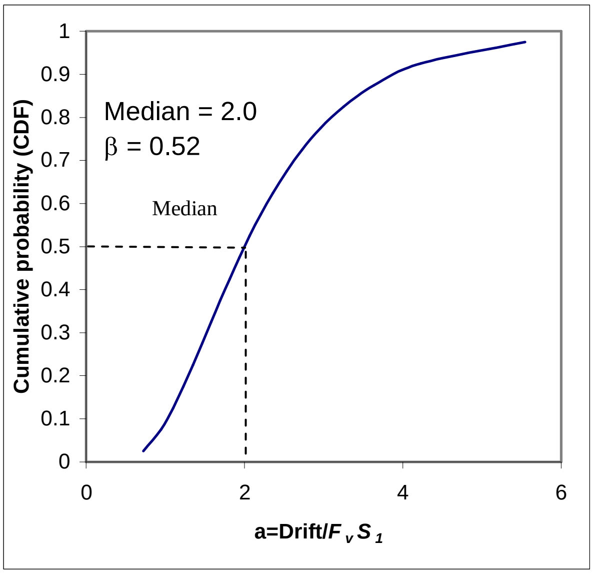


Figure 2: A cumulative function plot for the New Zealand concrete buildings.

It was shown that the results conformed quite well to a cumulative lognormal probability distribution with a median value of 2.0 and dispersion factor (lognormal coefficient of variation) of 0.52. Hence, the relationship between the median drift and the spectral acceleration can be mathematically expressed as:

$$\tilde{D}_D = 2.0(F_vS_1)_D \qquad (1)$$

in which $\tilde{D}_D$ = the median (50th percentile) drift demand as a percentage of the storey height, $(F_vS_1)_D$ = one second spectral acceleration. Inverting Equation (1), the expected value of ground motion demand needed to achieve a given median drift capacity can be calculated as:

$$(F_vS_1)_C = 0.5\tilde{D}_C \qquad (2)$$

where $\tilde{D}_{C\,=}$ expected drift capacity of the structure, which is difficult to determine precisely. Although full-scale experiments may give a good indication of the expected capacity, uncertainties are bound to be associated with this determined drift capacity. Acknowledging this, a lognormal distribution function was assumed for the drift capacity and a lognormal coefficient of variation β_c = 0.2 was used as suggested by Dutta [14].

When capacity and demand are merged in design, uncertainties of both components need to be taken into account. As explained earlier, the uncertainty in drift demand has a lognormal coefficient of variation of β_D = 0.52. When merging lognormal distributions [15], the resultant lognormal coefficient of variation can be calculated as:

$$\beta_{C/D} = \sqrt{\beta_C^2 + \beta_D^2 + \beta_U^2} \qquad (3)$$

where β_U = dispersion parameter to account for modelling uncertainty, taken here β_U = 0.2. Applying (3) gives $\beta_{C/D}$ = 0.6. By using a lognormal cumulative distribution that can be described by a lognormal variate ξ_β (where the median = 1 and the lognormal coefficient of variation, $\beta_{C/D}$ = 0.6), the distribution of ground motion demands needed to produce a given state of damage can be found by

$$F_vS_1 = 0.5\tilde{D}_C(\mathrm{DS})\xi_\beta \qquad (4)$$

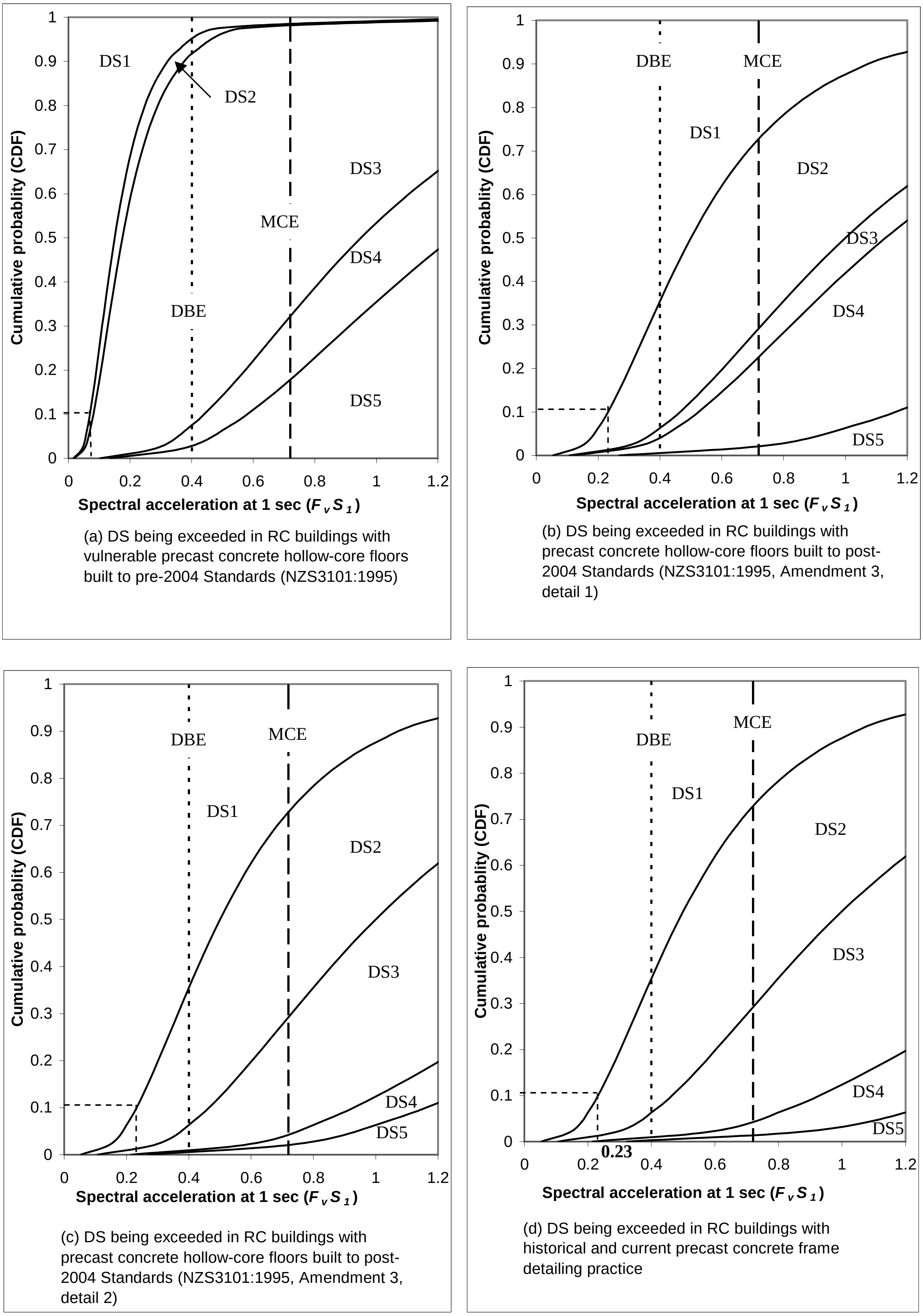
where $\tilde{D}_C$(DS) = the expected drift (in this case, the experimentally observed drift) corresponding to a given damage state (DS) as listed in Table 2.

2.4 Generation of Fragility Curves

Using the ground motion demand for a median drift capacity calculated from Equation (2) and the resultant lognormal coefficient of variation determined earlier, the probability of building response being within a given drift limit can be calculated. Replacing drift with damage states using Table 2 will then give the fragility curves, which show graphically the probability of different damage states being exceeded in an earthquake. For buildings with floor-frame connections designed to pre-2004 standards and post-2004 amendment (detailing 1 and detailing 2) and for similar buildings with perfect/no floors, fragility curves are shown in Figure 3. Two vertical lines are drawn at 0.4g and 0.72g to represent respectively the design basis earthquake (DBE) and the maximum considered earthquake (MCE) at Wellington, following the seismic hazard reported in the loading standard NZS4203:1992 [16]. The intersection of these vertical lines with the fragility curves gives the probability of different damage states for the corresponding seismic hazard.

Figure 3a shows that due to the poor performance of the precast hollow-core floor with old reinforcing and connection details only 2% of buildings with such details would be expected to sustain slight or no damage (within damage state DS2) during an MCE. The remaining 98% buildings would be expected to experience moderate to severe damage (above damage state DS2), of these some 32% would be expected to partially or entirely collapse requiring demolition of the building (above damage state DS4). Similarly it is also evident from Figure 3a that even under a DBE, only 4% of these buildings would escape damage whereas some 8% buildings may still be irreparably damaged or collapsed potentially leading to loss of life.

Figures 3b and 3c show the probability of different extent of damage if the buildings performance is classified in terms of the performances of the precast hollow-core floors with detailing 1 and detailing 2 of post-2004 amendment and the frame performance respectively. Figures 3b and 3c indicate that 70% of buildings with improved connection detailing might be expected to sustain either slight or no damage in an MCE. Figure 3b shows that 23% buildings with floor-frame connection detailing 1 would be expected to be severely damaged. On the other hand, probability of severe damage (DS4 or DS5) in an MCE for buildings with floor-frame connection detailing 2 or for similar buildings with perfect/no floor (i.e. damage contributed by the frame only) is only 4% as shown in Figures 3c and 3d. Under a DBE, 93% buildings with post-2004 floor-frame connection details might be expected to sustain repairable damage (see Figures 3b and 3c). This probability is the same in buildings with perfect/no floor (see Figure 3d) because the inter-storey drift corresponding to the DS2-DS3 boundary is the same in Table 2 for the improved floor-frame connection (both detailing 1



(a) DS being exceeded in RC buildings with vulnerable precast concrete hollow-core floors built to pre-2004 Standards (NZS3101:1995)

(b) DS being exceeded in RC buildings with precast concrete hollow-core floors built to post-2004 Standards (NZS3101:1995, Amendment 3, detail 1)

(c) DS being exceeded in RC buildings with precast concrete hollow-core floors built to post-2004 Standards (NZS3101:1995, Amendment 3, detail 2)

(d) DS being exceeded in RC buildings with historical and current precast concrete frame detailing practice

Figure 3: Fragility Curves for New Zealand multi-storey RC buildings related to the HAZUS damage states.

and detailing 2) and for buildings with perfect/no floor. On the other hand, probability of heavy damage leading to partial collapse of buildings with post-2004 connection Detail 1 would be 4%, while only 1% buildings with connection Detail 2 and buildings with perfect/no floor frame detailing would be expected to suffer heavy damage in case of a DBE. Again, this is attributable to the same DS3-DS4 boundary for these two cases in Table 2.

Comparison of Figure 3d with Figures 3b and 3c indicates that the fragility of buildings is not affected adversely by floors with improved connection detail, whereas comparison of Figures 3a and 3d informs that the inclusion of floors with the vulnerable pre-2004 connection detail render the building significantly more fragile. Therefore, the overall performance will be governed by the poor performance of the floor in pre-2004 buildings whereas the performance of the post-2004 buildings could be judged by the performance of either the floor (with detailing 2) or the frame as both of these components are found to be equally fragile.

3. FINANCIAL SEISMIC RISK ASSESSMENT FRAMEWORK

Communicating seismic vulnerability to decision makers is an important aspect of performance based earthquake engineering (PBEE). One such communication tool is Expected Annual Loss (EAL) which can be expressed in a dollar value. EAL incorporates the entire range of seismic scenarios, return rate, and expected damage into a median dollar loss. Though there are many methods of quantifying financial risk, EAL is especially useful to decision makers for cost-benefit analysis of design alternatives for new structures or seismic retrofit alternatives for existing structures. Moreover, EAL can easily be accounted for by including into operating budgets.

Recent research at the Pacific Earthquake Engineering (PEER) Center on seismic risk assessment has led to a mathematical expression in the form of a triple integral equation [4] that can be used to evaluate the probability of an arbitrarily chosen decision variable exceeding a prescribed limit. The interrelationships used in the triple integration link firstly seismic hazard to structural response, then response to damage, and finally damage to the decision variable. If the decision variable is expressed in terms of economic consequences, the triple integral equation can be used to estimate the total probable loss due to an earthquake. Dhakal and Mander [5] have extended the PEER framework formula to a quadruple integral by including time, thereby enabling the quantification of seismic risk in terms of EAL. The quadruple integral formulation is given as:

$$EAL = \int_0^1\int_0^1\int_0^1\int_0^1 L_R.dP[L_R|DM].dP[DM|EDP].dP[EDP|IM].df_a[IM] \tag{5}$$

in which, IM = intensity measure; f_a[IM] = annual probability of an earthquake of a given intensity IM; EDP = engineering demand parameter; DM = damage measure; L_R = loss ratio (i.e. decision variable); P[A|B] = shortened form of P[A ≥ a | *B*=b]; and dP[A|B] = derivative of the conditional probability P[A|B] with respect to A.

Equation (5) provides a foundation from which the following subtasks can be performed: evaluating the probability of seismic hazard; analysing structural fragility; damage assessment; and loss estimation. Implicit in the formula is a probabilistic analysis, which incorporates a number of uncertainties to be combined in accordance with the total probability theorem [15] as described by Equation (3).

As is evident from Figure 1, the Intensity measure (IM) used in this study is F_vS_1 (the spectral acceleration at 1 second). The EDP considered is maximum inter-storey drift, which can be associated with damage in a global sense in terms of partial/complete collapse and in a local sense in terms of yielding, spalling, and bar buckling. To quantify damage, damage states defined according to HAZUS [11] are adopted, classifying damage into 5 distinct categories, as summarized in Table 1. In order to relate EDP with the damage measure (DM), drifts causing different damage states are specified as listed in Table 2. For calculating EAL using Equation (5), two more variables need to be defined, namely loss ratio (L_R) and annual probability (f_a), and their correlation with one of the three previously defined parameters (IM, EDP and DM) need to be established. The interrelationships between f_a and IM and between L_R and DM are explained in the following sections.

4. ASSESSMENT OF HAZARD SURVIVAL PROBABILITY

4.1 Earthquake Recurrence Relationship

Note that the fragility curves shown in Figure 3 are plots of P[DM|IM] (which is the product of P[DM|EDP] and P[EDP|IM]) against IM (F_vS_1 in this study). In order to use these curves as a part of Equation (5), the horizontal axis needs to be annual probability (f_a) rather than the hazard intensity. Hence, it is necessary to define a relationship between the annual probability of earthquakes and their intensity.

Based on historical earthquake data, relationship between the peak ground acceleration (PGA) of earthquakes (denoted as a_g) with their annual probability of occurrence (f_a) has been established as:

$$a_g = \frac{a_g^{DBE}}{(475 f_a)^q} \tag{6}$$

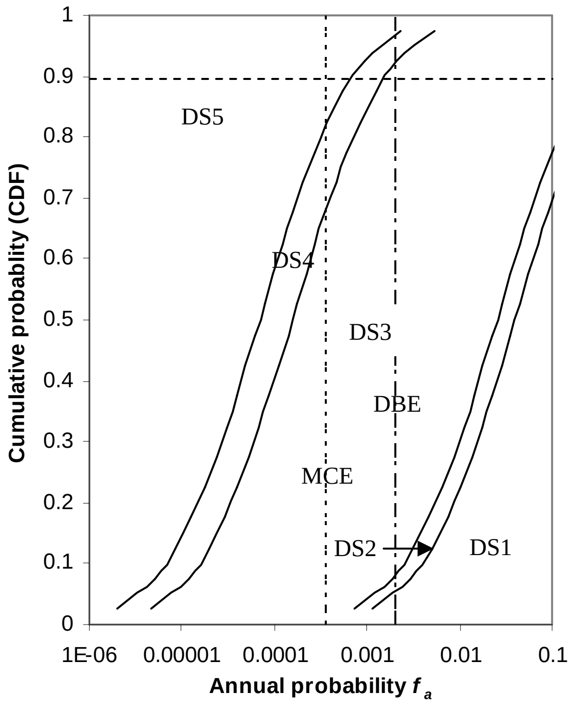
where a_g^{DBE} is the PGA of the DBE (10% probability of occurrence in 50 years) and q is an empirical constant found to be equal to 0.33 for seismic hazard in NZ [16].

As the IM used in this study is the spectral acceleration at 1 sec (F_vS_1), relationship between spectral acceleration and PGA is desirable to utilize Equation (6). In the constant velocity region of the design spectra, which spans through 1 sec and covers a range in which the natural periods of most structures are likely to fall, the equation of the spectral acceleration curve is:

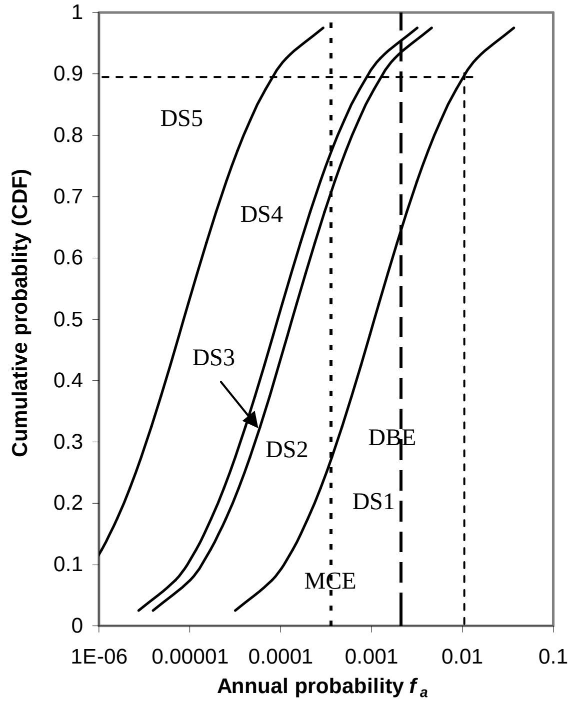
$$S_T = \frac{a_g}{T.S} \tag{7}$$

where T is the natural period of structures (in sec); S_T is the spectral acceleration at that period; and S is soil factor. Assuming firm soil for which the soil factor S is unity, the spectral acceleration at 1 sec period is hence equal to the PGA; i.e. $F_vS_1 = a_g$.

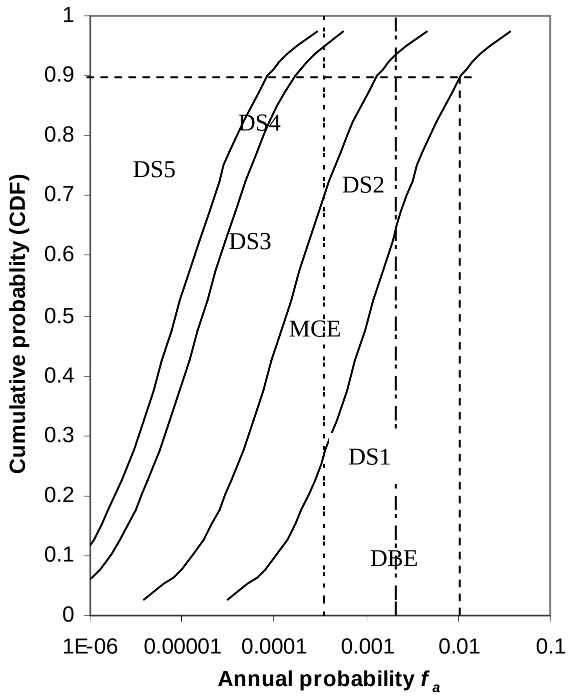
It is to be noted that, as investigated by Der Kiureghian [17], earthquakes are discrete, rather than continuous events, and should be modelled as a Poison process. In this case, the hazard-recurrence formula given above, though conservative, is not strictly correct when f_a > 0.01. In order to compensate for this shortcoming to some extent, the contribution of frequent earthquakes (i.e. f_a > 0.1) is not included in this study.



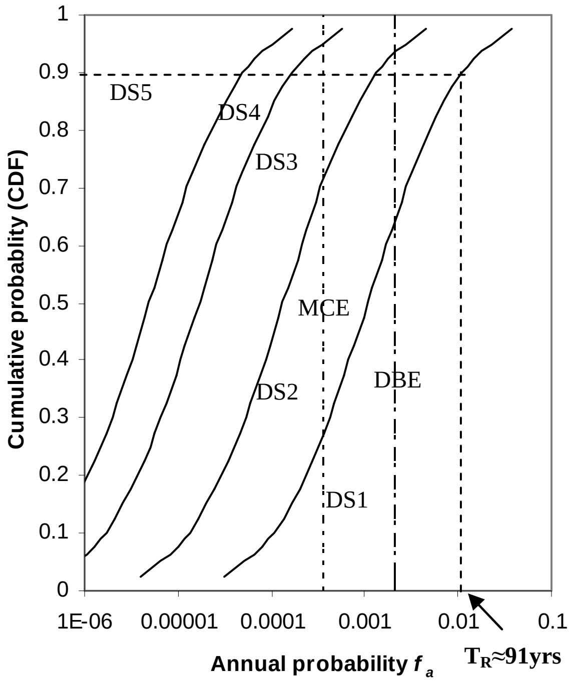
(a) DS not being exceeded (confidence) in RC buildings with vulnerable precast concrete hollow-core floors built to pre-2004 Standards (NZS3101:1995)



(b) DS not being exceeded (confidence) in RC buildings with vulnerable precast concrete hollow-core floors built to post-2004 Standards (NZS3101:1995, Amendment 3, detail 1)



(c) DS not being exceeded (confidence) in RC buildings with vulnerable precast concrete hollow-core floors built to post-2004 Standards (NZS3101:1995, Amendment 3, detail 2)



(d) DS not being exceeded (confidence) in RC buildings with historical and current precast concrete frame detailing practice

Figure 4: ***Hazard Survival Curves for New Zealand multi-storey RC buildings related to the HAZUS damage states.***

Table 3a Probability of not exceeding different damage states for buildings built to pre-2004 standards with vulnerable precast concrete hollow-core floors

P[DS ≤ DSi]					
f_a	i=1	i=2	i=3	i=4	i=5
0.1	0.7	0.78	1	1	1
0.01	0.22	0.3	1	1	1
0.001	0.02	0.04	0.85	0.93	1
0.0001	0	0	0.4	0.58	1
0.00001	0	0	0.06	0.14	1
0.000001	0	0	0	0	1

Table 4a Probability of being in a given damage state (confidence interval) for buildings built to pre-2004 standards with vulnerable precast concrete hollow-core floors

P[DS = DSi]					
f_a	i=1	i=2	i=3	i=4	i=5
0.1	0.7	0.08	0.22	0	0
0.01	0.22	0.08	0.7	0	0
0.001	0.02	0.02	0.81	0.08	0.07
0.0001	0	0	0.4	0.18	0.42
0.00001	0	0	0.06	0.08	0.86
0.000001	0	0	0	0	1

Table 3b Probability of not exceeding different damage states for buildings with improved connections built to the 2004 amendment (Detailing 1)

P[DS ≤ DSi]					
f_a	i=1	i=2	i=3	i=4	i=5
0.1	1	1	1	1	1
0.01	0.89	1	1	1	1
0.001	0.49	0.87	0.9	1	1
0.0001	0.1	0.44	0.51	0.92	1
0.00001	0	0.08	0.11	0.54	1
0.000001	0	0	0	0.12	1

Table 4b Probability of being in a given damage state (confidence interval) for buildings with improved connections built to the 2004 amendment (Detailing 1)

P[DS = DSi]					
f_a	i=1	i=2	i=3	i=4	i=5
0.1	1	0	0	0	0
0.01	0.89	0.11	0	0	0
0.001	0.49	0.38	0.03	0.1	0
0.0001	0.1	0.34	0.07	0.41	0.08
0.00001	0	0.08	0.03	0.43	0.46
0.000001	0	0	0	0.12	0.88

Table 3c Probability of not exceeding different damage states for buildings with improved connections built to the 2004 amendment (Detailing 2)

P[DS ≤ DSi]					
f_a	i=1	i=2	i=3	i=4	i=5
0.1	1	1	1	1	1
0.01	0.89	1	1	1	1
0.001	0.49	0.86	1	1	1
0.0001	0.09	0.44	0.84	0.92	1
0.00001	0	0.08	0.4	0.54	1
0.000001	0	0	0.06	0.12	1

Table 4c Probability of being in a given damage state (confidence interval) for buildings with improved connections built to the 2004 amendment (Detailing 2)

P[DS = DSi]					
f_a	i=1	i=2	i=3	i=4	i=5
0.1	1	0	0	0	0
0.01	0.89	0.11	0	0	0
0.001	0.49	0.37	0.14	0	0
0.0001	0.09	0.35	0.4	0.08	0.08
0.00001	0	0.08	0.32	0.14	0.46
0.000001	0	0	0.06	0.06	0.88

Table 3d Probability of not exceeding different damage states for ideal buildings with perfect/no floor; i.e. damage contributed by the frame only

P[DS ≤ DSi]					
f_a	i=1	i=2	i=3	i=4	i=5
0.1	1	1	1	1	1
0.01	0.89	1	1	1	1
0.001	0.49	0.87	1	1	1
0.0001	0.09	0.44	0.84	0.95	1
0.00001	0	0.08	0.4	0.65	1
0.000001	0	0	0.06	0.19	1

Table 4d Probability of being in a given damage state (confidence interval) for ideal buildings with perfect/no floor; i.e. damage contributed by the frame only

P[DS = DSi]					
f_a	i=1	i=2	i=3	i=4	i=5
0.1	1	0	0	0	0
0.01	0.89	0.11	0	0	0
0.001	0.49	0.38	0.13	0	0
0.0001	0.09	0.35	0.4	0.11	0.05
0.00001	0	0.08	0.32	0.25	0.35
0.000001	0	0	0.06	0.13	0.81

4.2 Hazard Survival Curves

Fragility curves of Figure 3 can now be re-plotted by changing the horizontal axis from IM to f_a using the earthquake recurrence relationship established earlier. Such curves are called hazard-survival curves and they show the probability of damage being within a limit state when an earthquake of a given annual probability strikes. Figures 4a-4d show the hazard survival curves for the buildings with precast floors designed to pre-2004 standards and post-2004 amendment (detailing 1 and detailing 2) and similar buildings with perfect/no floor so that the performance of the buildings is governed by the seismic frames. Two vertical lines representing the annual probabilities of DBE (f_a ~ 0.002) and MCE (f_a ~ 0.0004) are also shown in the plots for reference. The intersections of any vertical line through a value of f_a with the hazard survival curves give the probability of these damage states not being exceeded in earthquakes of that annual probability of occurrence. Thus obtained damage state survival probabilities in earthquakes of different frequencies are shown in Tables 3a-3d for buildings with the three different floor-frame connection details and an ideal building with perfect/no floor. Similarly, Tables 4a-4d show the probabilities of being in a given damage state (confidence interval) for the four cases. For example, the second row in Table 3a means that if an earthquake of annual frequency of 0.01 (i.e. return period of 100 years) strikes, the probability of DS1 not being exceeded in buildings with the vulnerable pre-2004 connection detailing is 22%; and the corresponding probabilities for other damage states (DS2 and DS3) are 30% and 100% respectively. Similarly the second row of Table 4a means that when an earthquake with an annual frequency of 0.01 (i.e. return period of 100 years) strikes, there is a 22% chance that the damage state of these buildings will be DS1, 8% chance that the damage will be in the range of DS2 and so on.

5. FINANCIAL IMPLICATION OF EARTHQUAKES

5.1 Loss Model

To quantify financial loss, a loss model must be established to relate damage measure (DM) to a dollar value. In this study, the financial implication of each damage state is represented by a *loss ratio (*L_R*)*, which is the ratio of the cost necessary to restore the structure to full working order to the replacement cost. Deciding the cost implication of each damage state is a subjective process and the accuracy of the decided value will depend largely on the amount of time devoted to researching repair costs and their variation by extent of damage, location of building, etc.

Table 5 Loss ratios for different damage states

	DS1	DS2	DS3	DS4	DS5
Likely Range	0	0.05-0.15	0.2-0.4	1.0-1.2	1
Assumed L_R value	0	0.1	0.3	1	1

The assumed values and likely range of loss ratios for different damage states are shown in Table 5. As no damage or repair is expected in pre-yield damage state DS1, no financial loss is incurred and the loss ratio for DS1 is therefore zero. Loss ratio for DS2 is likely to fall between 0.05 and 0.15 to account for minor repairs due to slight but tolerable damage, and L_R = 0.1 is assumed for DS2. The loss ratio for DS3 may vary from 0.2 to 0.4 for repairing the incurred moderate damage to restore functionality, and a representative value of 0.3 is adopted in the present analysis. “Irreparable damage” under DS4 demands complete replacement as repair may be uneconomic; hence the loss ratio of 1 is used here. Similarly for DS5, which is complete failure/collapse the value of loss ratio is 1.

It has been shown [5] that the financial risk is sensitive to the values of loss ratios, especially L_R for DS2 and DS3. Hence, good judgement should be applied in deciding these values. However, the objective of this study is to compare the financial risk of different detailing schemes and a constant set of L_R values will not have considerable impact on the final comparative outcome.

5.2 Probable Loss in an Earthquake

Using the assigned loss ratios, the contribution of different damage states to the financial loss can be estimated. Table 6 lists the probable financial loss (as fraction of the total replacement cost) due to different damage states when earthquakes with annual frequencies of 0.1, 0.01, 0.001, 0.0001, and 0.00001 strike. The values in Table 6 are the product of the probability of being in a given damage state in earthquakes of different annual frequencies (obtained from corresponding Tables 4a-4d) and the assumed loss ratio for the corresponding damage state (obtained from Table 5). Graphical versions of Table 6 (i.e. economic hazard probability curves) are shown in Figures 5a-5d, which exhibit the contributions of different damage states and the total probable loss in the form of bar charts.

As expected, DS1 does not incur any financial loss as it does not need any repair. Similarly, the financial loss incurred by earthquakes of 0.1 or higher annual probability in case of buildings designed and built to post-2004 standards is also nil as such frequent events do not incur any damage requiring repair or replacement (DS2 or higher damage category). However some financial loss (up to 7% of the total cost) is expected due to repairable damage in buildings with vulnerable detailing of pre-2004 standards even by smaller earthquakes of 0.1 or higher annual probability. As shown in Table 5, the loss ratio L_R is higher for DS4 and DS5 than for other damage states. As confidence intervals of higher damage states are multiplied by higher loss ratio, the higher damage-states contribute more to the probable loss although the likelihood of the earthquake-induced damage falling into these severer categories is not high. Again in case of buildings designed to pre-2004 standards, repairable moderate damage (DS3) contributes most to the financial loss when earthquakes of 0.001 or higher probability (i.e. with return period of 1000 years or less) strike.

The total financial loss due to earthquakes of a given probability shown in the last column of Table 6 is the sum of the contributions of the five damage states. Figure 6 plots the total loss ratio against the annual probability for the four cases. These curves give information on what would be the financial loss if an earthquake of a given annual probability strikes once. As expected the larger and rarer the event the greater the financial loss. Conversely for frequent, but low intensity events, the single-event loss is small.

Two vertical lines corresponding to DBE and MCE are also shown in Figure 6. It is evident from Figure 6 that a building with pre-2004 connection details is likely to lose about 30% and 50% of its value due to damage incurred by a DBE and an MCE, respectively. Even a small earthquake with 0.1 annual frequency (return period of 10 years) is likely to incur 7% loss to these buildings. Obviously, maintenance of such buildings in a seismic zone would be costly. On the other hand, as can be seen in Figure 6, buildings with improved post-2004 detailing will remain almost intact (losing only

Table 6 Probable financial loss analysis

	f_a	L_R[DS1]	L_R[DS2]	L_R[DS3]	L_R[DS4]	L_R[DS5]	Total L_R
a) Pre-2004 Standard [Matthews]	0.1	0	0.004	0.066	0	0	0.07
	0.01	0	0.004	0.21	0	0	0.214
	0.001	0	0.001	0.243	0.06	0.07	0.374
	0.0001	0	0	0.12	0.135	0.42	0.675
	0.00001	0	0	0.018	0.06	0.86	0.938
	0.000001	0	0	0	0	1	1
b) Post-2004 (Detailing 1) [Lindsay]	0.1	0	0	0	0	0	0
	0.01	0	0.0055	0	0	0	0.0055
	0.001	0	0.019	0.009	0.075	0	0.103
	0.0001	0	0.017	0.021	0.3075	0.08	0.4255
	0.00001	0	0.004	0.009	0.3225	0.46	0.7955
	0.000001	0	0	0	0.09	0.88	0.97
c) Post-2004 (Detailing 2) [MacPherson]	0.1	0	0	0	0	0	0
	0.01	0	0.0055	0	0	0	0.0055
	0.001	0	0.0185	0.042	0	0	0.0605
	0.0001	0	0.0175	0.12	0.06	0.08	0.2775
	0.00001	0	0.004	0.096	0.105	0.46	0.665
	0.000001	0	0	0.018	0.045	0.88	0.943
d) Ideal (perfect/no floor) Frame detailing	0.1	0	0	0	0	0	0
	0.01	0	0.0055	0	0	0	0.0055
	0.001	0	0.019	0.039	0	0	0.058
	0.0001	0	0.0175	0.12	0.0825	0.05	0.2700
	0.00001	0	0.004	0.096	0.1875	0.35	0.6375
	0.000001	0	0	0.018	0.0975	0.81	0.9255

0.5% of its value) in a once in 100 years earthquake (f_a ~ 0.01). In a DBE and an MCE, these buildings with detail 1 will incur a loss of about 5% and 22% respectively, which are drastically smaller than those for pre-2004 buildings. This loss will further reduce for buildings with detail 2 being about 3% and 13% in DBE and MCE respectively. As can be seen in Figure 6, these values are very close to those for idealised buildings with perfect/no floor (i.e. money needed to repair frame damage only); indicating that the floor with improved post-2004 detailing do not cause any additional financial burden in terms of maintenance.

6. SEISMIC ANNUAL FINANCIAL RISK

6.1 Calculation of Expected Annual Loss (EAL)

At this point, each component of the probabilistic analysis process has been established. Relationships have been generated to relate IM to EDP (Figure 2), EDP to DM (Tables 1 and 2), and DM to L_R (Table 5). The total expected annual loss can now be calculated using Equation 5 by integrating the loss ratio over all possible annual frequencies of the seismic hazard; i.e. between 0 and 1. This general equation in continuous form can be expressed as:

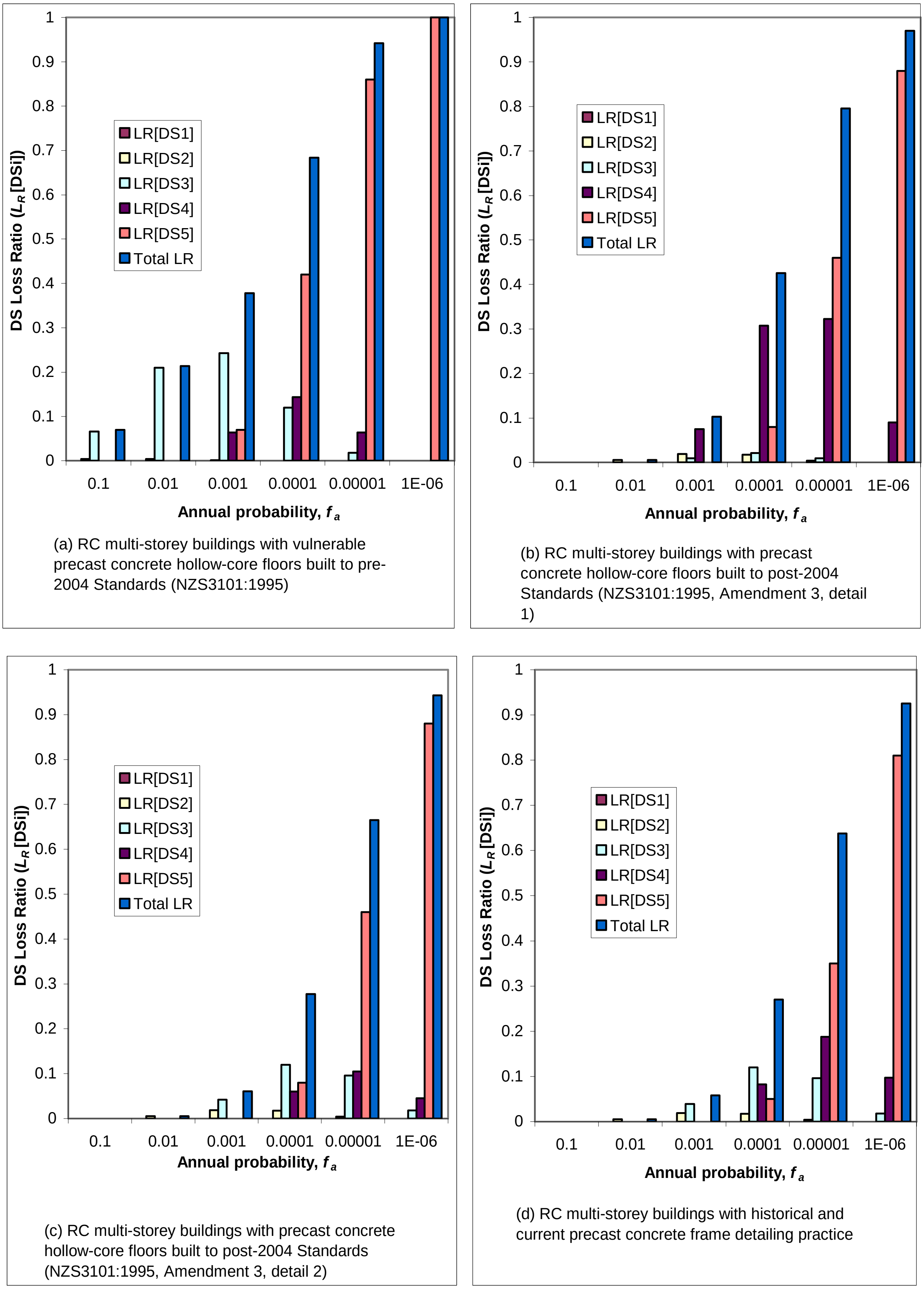
$$\text{EAL} = \int_0^1 L_R \, df_a \tag{8}$$

In discrete form, the expected annual loss (EAL) can be calculated as:

$$\text{EAL} = \sum_{all\, l_{r,i}} \left(\frac{l_{r,i} + l_{r,i+1}}{2} \right) \left(f_a[L_R = l_{r,i}] - f_a[L_R = l_{r,i+1}] \right) \tag{9}$$

in which $f_a[L_R=l_r]$ is the annual probability of the loss ratio being equal to a given value l_r which can be obtained from the economic hazard probability curves (Figure 6). Table 7 shows the annual loss of the buildings with the three different floor-frame connection details and similar building governed by the frame. First, the probable loss due to earthquakes of annual probability within a range is calculated which is the area subtended by the economic hazard curves (Figure 6) between two points on the x-axis. Then the losses contributed by the earthquakes with different ranges of probability are added together to obtain the total expected annual loss (EAL). It can be noted that the annual probability is plotted in logarithmic scale in Figure 6, and the absolute value of the interval between any two points on the x-axis decreases by an order of ten towards the left. Accordingly, the absolute value of the area covered is also decreasing rapidly in that direction (i.e. direction of decreasing probability) in spite of a higher value of the loss ratio. As can be observed from Table 7, the EAL of the buildings built to post-2004 improved connection detailing is approximately 5%-7% of that of buildings built to the vulnerable pre-2004 detailing. The large difference in the total loss ratio between pre- and post-2004 buildings with precast concrete floors for different earthquakes can also be noted in Figure 6.

As mentioned earlier, this model overestimates the EAL by over-emphasising the contribution of frequent events (fa > 0.01; i.e. return period of less than 100 years). The error can be compensated by truncating the data above a certain threshold. This threshold is found by locating the IM at which there will be no damage, say with 90% confidence. For example, to induce damage to the ideal buildings with perfect/no floor, earthquakes with $F_vS_1 < 0.23$g (return period of approximately 91 years) will have 90% probability of not inducing any damage (see Figures 3d and 4d). Contribution to EAL of earthquakes below this threshold, if not considered, will have a considerable effect on the final result. The EAL for these ideal multi-storey RC buildings with perfect/no floor is found to be about 34% lower after truncating the data below this threshold. The reduction of EAL by ignoring the contribution of earthquakes below similarly decided thresholds for buildings designed after the 2004 amendment is 24% and 34% for detailing 1 and detailing 2, respectively.



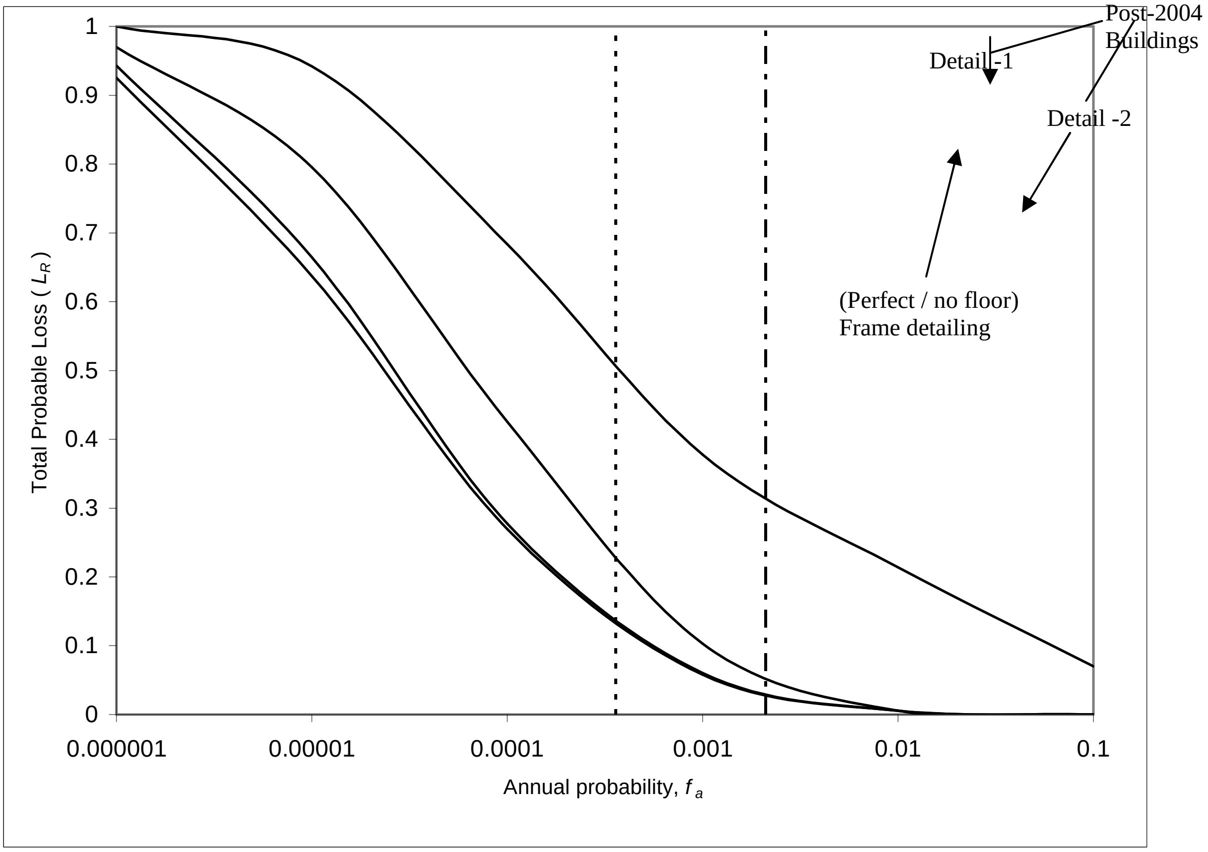
(a) RC multi-storey buildings with vulnerable precast concrete hollow-core floors built to pre-2004 Standards (NZS3101:1995)

(b) RC multi-storey buildings with precast concrete hollow-core floors built to post-2004 Standards (NZS3101:1995, Amendment 3, detail 1)

(c) RC multi-storey buildings with precast concrete hollow-core floors built to post-2004 Standards (NZS3101:1995, Amendment 3, detail 2)

(d) RC multi-storey buildings with historical and current precast concrete frame detailing practice

Figure 5: ***Econcomic Hazard Probability Curves (Bar Charts) for New Zealand multi-storey RC buildings related to the HAZUS damage states.***

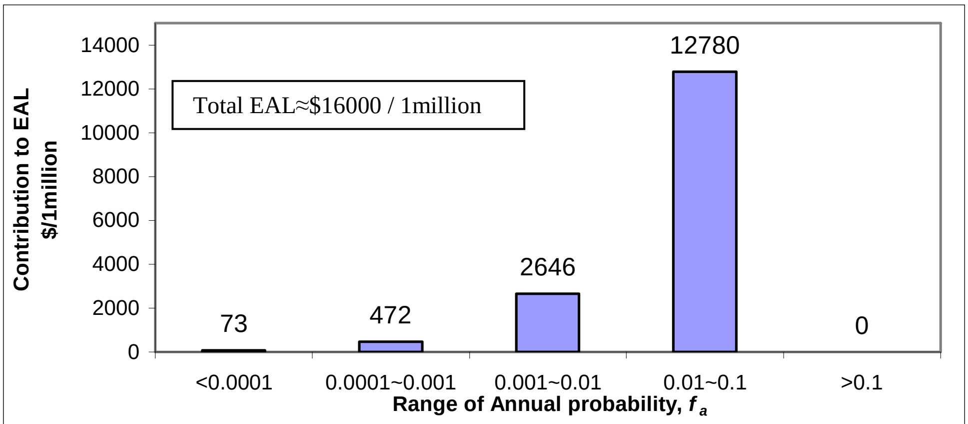


MCE

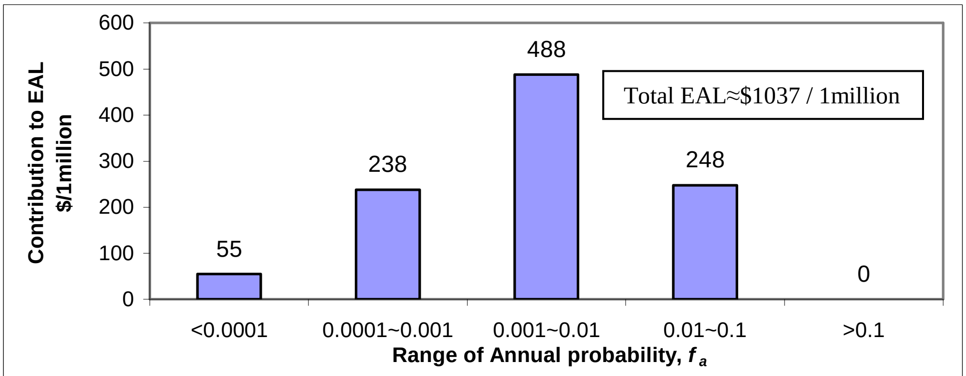
Figure 6: Econcomic Hazard Probability Curves for New Zealand multi-storey RC buildings.

Table 7 Annual financial risk for buildings

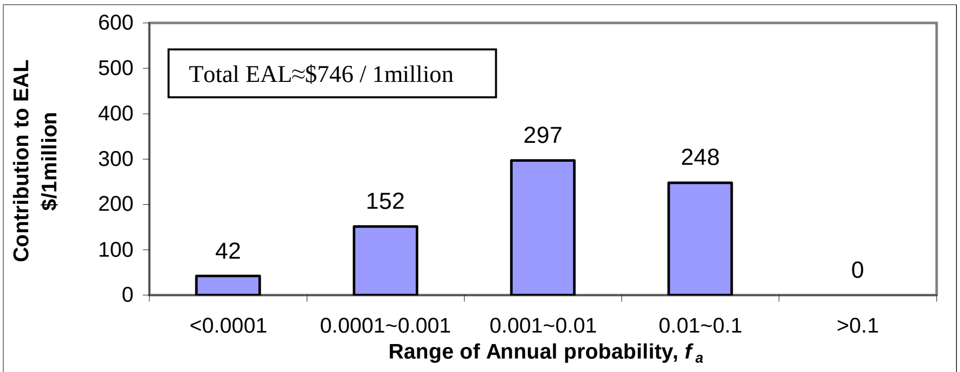
f_a	EAL (per $1 million)							
	Pre-2004 standards [Matthews]		Post-2004 (Detailing 1) [Lindsay]		Post-2004 (Detailing 2) [MacPherson]		Ideal (perfect/no floor) Frame detailing	
	L_R	ΔEAL	L_R	ΔEAL	L_R	ΔEAL	L_R	ΔEAL
0.1	0.07		0		0		0	
		12780		248		248		248
0.01	0.214		0.0055		0.0055		0.0055	
		2646		488		297		286
0.001	0.374		0.103		0.0605		0.058	
		472		238		152		148
0.0001	0.675		0.4255		0.2775		0.27	
		72.6		55		42.4		40.8
0.00001	0.938		0.7955		0.665		0.6375	
		8.72		8		7.24		7.03
0.000001	1		0.97		0.943		0.9255	
Total EAL		16000		1037		746		729



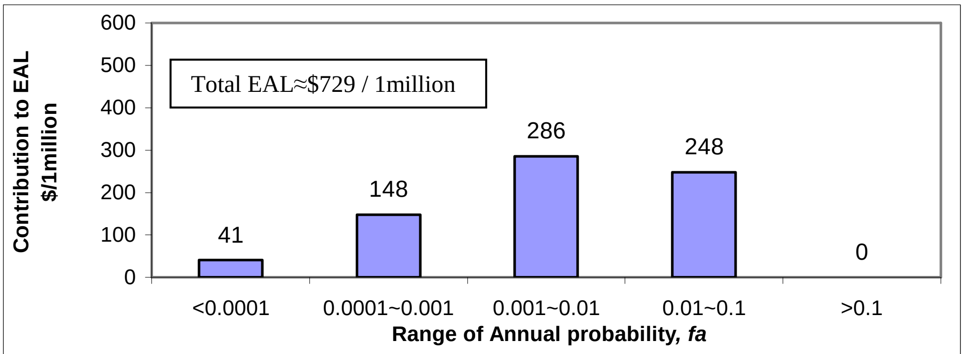
(a) RC multi-storey buildings with vulnerable precast concrete hollow-core floors built to pre-2004 Standards (NZS3101:1995)



(b) RC multi-storey buildings with precast concrete hollow-core floors built to post-2004 Standards (NZS3101:1995, Amendment 3, detail 1)



(c) RC multi-storey buildings with precast concrete hollow-core floors built to post-2004 Standards (NZS3101:1995, Amendment 3, detail 2)



(d) RC multi-storey buildings with historical and current precast concrete frame detailing practic

Figure 7: ***Annual financial risk for New Zealand multi-storey RC buildings due to earthquakes of different probability.***

The buildings with vulnerable precast concrete floors with pre-2004 details, however, will have an increase in EAL if 90% confidence level is considered for truncation. As can be noticed in Figure 4a, the horizontal line through 0.9 in the vertical axis (i.e. indicating a 90% confidence) does not intersect the hazard survival curve separating DS1 and DS2 within the plotted range of annual probability. In other words, the annual frequency of earthquakes having a 90% probability of no damage is more than 0.1, data above which were not included in the calculation. Obviously, if the contribution of earthquakes with annual frequency more than 0.1 is included, the ultimate value of EAL would increase significantly. The threshold frequency for the other three cases is less than 0.1 and hence the truncation will reduce the EAL, which otherwise includes the frequency range from 0.000001 to 0.1. This further widens the gap between the financial risk of buildings with vulnerable pre-2004 and improved post-2004 connection details.

Note in Table 7 that EAL of ideal buildings with perfect/no floor is in the same range as that of the buildings with precast concrete hollow-core floors built to post-2004 connection detailing 2. This demonstrates the effectiveness of the recommendations made in the Amendment No. 3 of NZS 3101: 1995 regarding seating and connection details of precast concrete floors used in moment-resisting reinforced concrete frame buildings.

6.2 Implications to Owners and Insurers

A vertical ordinate of the economic hazard probability curves (Figure 6) gives the total probable loss of a building due to earthquakes for a given annual probability. Hence, they represent the financial risk to owners of individual buildings. Evidently, smaller and more frequent events pose a small risk to owners of buildings with post-2004 improved floor-frame connection details. Consequently, owners may be prepared to bear the risk of these frequent earthquakes by themselves. In the worst case, they may need to spend a small sum (less than 1% of the building value) to repair the damage (if any) incurred if and when these moderate earthquakes strike. On the other hand, the consequences of rarer but stronger earthquakes may be disastrous, often incurring 50% or more loss thereby rendering the repair uneconomical, necessitating replacement. Building owners would obviously be more inclined to pass this risk to insurers.

Note that the insurer's risk encompasses all insured buildings and all possible hazards. In other words, the integration of the economic hazard curve (Figure 6) represents insurer's risk. As EAL is the area subtended by the economic hazard curve, it represents insurer's risk and is directly related to an annual insurance premium for a building if all levels of seismic hazards are covered. The contribution of earthquakes of different frequency ranges to the total EAL is also graphically depicted in Figures 7a-d. Looking at the trend in Table 7 and Figures 7a-d, it is apparent that the earthquakes with annual frequencies smaller than 0.0001 (return period of more than 10000 years) will pose negligible financial risk. It is the more frequent and smaller events that pose more financial risk, and the large earthquakes amount to very small risks due mainly to their very small annual frequency of occurrence (long return period).

As is evident in Table 7 and Figure 7a, the total annual loss (i.e. the financial risk posed by all earthquakes) of the buildings with vulnerable precast concrete hollow-core floors built to pre-2004 standards amounts to about 1.6% of the replacement cost. In other words, the expected annual financial loss is $16000 per $1 million of building cost. 80% of this value corresponds to the risk posed by frequent but modest size earthquakes with an annual frequency in the range between 0.01 and 0.1 (i.e. return periods between 10 and 100 years). On the other hand, only 25% of the annual financial loss expected of the buildings with improved post-2004 connection using detailing 1 (approximately $1037 per $ 1 million of building cost) corresponds to the risk posed by frequent but modest size earthquakes (see Table 7 and Figure 7b). Similarly, as can be seen in Table 7 and Figure 7c, buildings with floor-frame connection detailing 2 and ideal buildings with perfect/no floor are expected to undergo even lesser annual financial loss of approximately $730-$750 per $1million of building cost (i.e. 0.07% of the replacement cost) and 35% of this value corresponds to the risk posed by frequent but modest size earthquakes.

Insurers would not be so concerned about the small risk posed by these large and rare events as they themselves would re-insure. The loss to owners, however, would be untenable. That is why most insurance policies are targeted to cover the rarer and bigger hazards. In contrast, the smaller and more frequent events will pose a small risk to the individual owners but a significant collective risk to the insurers. If these frequent hazards are excluded from the insurance policy, the EAL and consequently the annual insurance premium will reduce significantly. From an insurance point-of-view, the risk of these smaller and more frequent events should ideally be carried by the owner. This can be achieved by setting an appropriate deductible to the policy and thus keeping the remainder of the insured risk affordable for the owners. Obviously, a higher deductible reduces the insurance premium.

7. CONCLUSIONS AND RECOMMENDATIONS

Fragility curves drawn based on results of full-scale tests on RC frame with precast concrete hollow-core floor slabs have been used to estimate annual financial loss. Expected annual loss (EAL) has been calculated by using a generalised probabilistic financial risk assessment methodology for buildings with precast concrete hollow-core floors designed and built to vulnerable pre-2004 detailing practice in NZ and the two types of improved connection details recommended in the 2004 Amendment No. 3 to NZS3101:1995. The structural performance, fragility, hazard-survival probability and the associated financial risk of buildings with these three floor-frame connection details are compared with each other and also against those of an ideal seismic frame building with perfect/no floor to realize the weakness imparted on the building by the floor with different connection detail.

It is concluded that the seismic performance of precast hollow-core floors in buildings designed and built to pre-2004 standards is vastly inferior to the performance of seismic frames. The floor-frame connection of these older structures may be the weakest link and will dictate the extent of losses for such buildings. On the other hand, improving the floor-frame connection detail according to the NZS3101:1995 Amendment No. 3 brings the overall building performance on par with the frame performance. It indicates that the precast floor with improved post-2004 detailing do not noticeably weaken the building. It is found that the buildings with precast floor designed to pre-2004 standards are likely to incur about 30% and 50% loss in a DBE (10% in 50 years event) and an MCE (2% in 50 years event) respectively, whereas the improvement in the connection details according to 2004 amendment will reduce the total probable loss to 3% in a DBE and 13% in an MCE. The EAL of precast concrete structures with hollow-core floor systems built to pre-2004 standards is found to be very high; in the order of $16,000 per $1million asset value, whereas the annual financial risk of similar buildings with improved post-2004 connection details is only about 7% of that of buildings with pre-2004 details.

Based on the discussions presented herein, it can be concluded that very large earthquakes pose almost negligible financial risk due to their very low probability of occurrence although structures are likely to partially or completely collapse if rare earthquakes of such magnitude strike. On the other hand, smaller earthquakes may only cause repairable minor-moderate damage to structures, but these earthquakes pose a big risk as they are likely to strike more often. Calculations showed that earthquakes with a return period between 10 and 100 years would contribute approximately 25% to the annual financial risk in case of RC buildings with precast floors with the improved post-2004 connection details, whereas the share of these frequent earthquakes is 80% in case of buildings with precast concrete hollow-core floors designed to pre-2004 standards. Thus, the not-so-high risk posed by frequent and moderate earthquakes may be born by the owners of post-2004 buildings, and the risk posed by rare and strong earthquakes may easily be covered by a low-premium insurance policy. However, owners of pre-2004 buildings with precast concrete hollow-core floors may need to insure their buildings even for smaller and more frequent earthquakes, and will subsequently pay a heavy insurance premium.

While this study has given interesting and useful qualitative information on the relative performance and financial implications of the different floor-frame detailing schemes, the dollar values obtained are only representative and are not precise because of the assumptions and approximations that have been made in the process. Although variations in the capacity and demand and the modelling uncertainty have been quantitatively incorporated in the form of corresponding lognormal coefficients of variation, uncertainties in the assumed loss model have not been accounted for. The values assigned in this study to loss ratios and drift ratios for different damage states are somewhat subjective. EAL is very sensitive to the loss ratio corresponding to different damage states; especially those for DS2 and DS3. Hence, more realistic interrelationship between the loss ratio and damage measure is needed. Nevertheless, the objective of this study is to investigate relative performance of the three different connection details, and a constant set of L_R values for different damage states across the three cases will have little effect on their relative position. Notwithstanding, future studies should try to establish more robust damage model and loss model and investigate their uncertainties so that they could be accounted for in estimating the financial risk.

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