

A refinement to earthquake vulnerability models implemented in the EQRM: correcting underestimation of the damage probability for nonstructural acceleration-sensitive components

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ABSTRACT: Geoscience Australia (GA) has developed the Earthquake Risk Model (EQRM) as an open source software package for earthquake hazard and risk assessment. In the EQRM, the likelihood of physical damage states for buildings and direct economic loss from the damage to structural and non-structural building components are estimated using fragility and vulnerability models, respectively. The methodology implemented in the EQRM to compute the likelihood of physical damage states and economic loss is similar to the HAZUS methodology, which is based on the capacity spectrum method (CSM) applied to a generalized Single-Degree-Of-Freedom (SDOF) model of the building. One limitation of the current approach is identified, which is the underestimation of the damage probability for non-structural acceleration-sensitive (NSA) components with a consequent underestimation of economic loss. This underestimation is found to be more problematic for larger ground shaking intensities. To overcome the limitation of the current methodology, time history analysis of a SDOF system with an elliptical hysteresis is performed and regression analysis is conducted to relate structural response and input ground moition parameters to maximum absolute acceleration. The estimated maximum absolute acceleration is then used in computing damage probability for NSA components instead of the spectral acceleration of the performance point based on the CSM. The effect of the new fragility of the NSA component and the ressulting vulnerability models are highlighted and discussed in a probabilistic risk assessment for a building portfolio in Newcastle, Australia.

1 INTRODUCTION

The vulnerability model is a key component of an earthquake risk assessment, along with exposure data and a hazard model. Vulnerability models are used to estimate the likelihood of physical damage states and the economic loss at a certain ground shaking intensity (e.g., peak ground acceleration). Models defining the likelihood of physical damage states are often called fragility models.

Geoscience Australia (GA) has developed the Earthquake Risk Model (EQRM), which is an open source software package for probabilistic earthquake hazard and risk assessment (Robinson et al., 2005, Robinson et al., 2006). In the EQRM, building response is computed using the capacity spectrum method (CSM) applied to a generalised Single-Degree-Of-Freedom (SDOF) model of the building. The computed building response or so-called "performance point" is subsequently used in computing the likelihood of physical damage states and the economic loss associated with three building components: structural, non-structural drift-sensitive (NSD) and non-structural acceleration-sensitive (NSA) components.

In computing the fragility for the NSA component, the spectral acceleration of the performance point is used based on the assumption that the spectral acceleration is equal to average upper-floor acceleration of a building (Kircher et al., 1997). But the spectral acceleration of the capacity curve becomes flat after the ultimate capacity point is reached as shown in Figure 1a. Therefore, the damage probability for the NSA component becomes constant after the ultimate point, no matter how large the level of input ground motion is, which seems illogical for general buildings.

To overcome this limitation, in this study the maximum absolute acceleration of a generalised SDOF model of a building is assumed to be equal to the average upper-floor acceleration of the building. In order to estimate the maximum absolute acceleration of the SDOF system, time history analysis of the

SDOF system with an elliptical hysteresis is performed using a large number of ground motions. Regression analysis is conducted to predict the maximum absolute acceleration with the structural response and the input ground motion parameters which can be obtained from or used as input to the CSM. The predicted absolute acceleration is then used in computing damage probability for NSA components instead of the spectral acceleration of the performance point based on the CSM. The new NSA component fragility model and the resulting vulnerability model are compared with the current model, and the effect of the modifications is highlighted and discussed in a probabilistic risk assessments for a building portfolio in Newcastle, Australia.

2 FRAGILITY AND VULNERABILITY MODELS IN THE EQRM

As of this writing there are two ways of defining vulnerability models in the EQRM: the first is to input parameter values required for an engineering approach primarily based on the CSM, and the second is to directly input felt intensity-based vulnerability models.

In the engineering approach, the building response (or performance point) is computed using the CSM applied to a generalised SDOF model of the building, which is similar to the HAZUS methodology (NIBS, 2003). The computed building response is subsequently used in computing the likelihood of physical damage states of three building components: structural, NSD and NSA. The economic loss of each component is computed as the product of the likelihoods for each damage state and the corresponding building repair cost. In HAZUS the building repair cost for a given damage state for each of the three components is assumed to be dependent on the HAZUS occupancy type. The total loss is estimated as the sum of the loss of each component.

The likelihood of physical damage state, represented as a conditional probability of being in or exceeding a certain damage state, ds, given the seismic demand parameter, s, is defined as:

$$P(DS \ge ds | S = s) = \Phi\left[\frac{1}{\beta_{ds}} \ln\left(\frac{s}{m_{ds}}\right)\right]$$
(1)

where m_{ds} and β_{ds} are the median and logarithmic standard deviation of the threshold of damage state, ds, respectively and Φ is the cumulative standard normal distribution function.

The seismic demand for both the structural and the NSD components is the spectral displacement of the performance point. For the NSA component fragility, the spectral acceleration of the performance point is used based on the assumption that this spectral acceleration is equal to average upper-floor acceleration. For a SDOF system, this assumption is valid while the system remains in the elastic range or the system has zero or very low damping. However, the assumption becomes invalid once the structure enters the inelastic range.

Figure 1b shows the NSA component fragility curve and the vulnerability curve of HAZUS buildingoccupancy type S3-IND2, Pre-code, which represents a Steel Light Frame building for a Light Industrial factory designed to pre-code standards. For the computation of the NSA fragility curves, a large number of ground motions are simulated using the Toro et al. (1997) ground motion model for a scenario event of M_w 6.5 and Joyner-Boore distance 10 km. A single set of magnitude and distance is used along with mean capacity curve for the building and site amplification is ignored to obtain smooth curves.

The values of the NSA component fragility and vulnerability curves become capped despite increasing ground motion intensity due to the flattening of the NSA component fragility curve, as discussed above. To overcome this limitation, in this study the maximum absolute acceleration of the SDOF system is assumed to be equal to the average upper-floor acceleration.



Figure 1. a) Capacity curve, and b) NSA component fragility (blue) and vulnerability (black solid) curves for S3-IND2, Pre-code

3 TIME HISTORY ANAYLSIS OF SDOF SYSTEM AND REGRESSION ANALYSIS

In order to overcome the limitation of the current CSM-based approach, time history analysis of the SDOF system is performed to estimate the maximum absolute acceleration with input and/or output of the CSM. The SDOF system is modelled using the parameter values for the capacity curve, and the hystereis of the SDOF system is modelled to follow an elliptic hysteresis (Karaca and Luco, 2008) as shown in Figure 2. A suite of 30 records (Vamvatsikos and Cornell, 2006) is used for the input ground motions, and each of the input ground motions is scaled up until response displacement reaches 10 times the spectral displacement at the ultimate capacity point.



Figure 2. Hysteresis curve for S3-IND2, Pre-code under one of the recorded Northridge earthquake (1994) records

For each time history analysis, maximums of relative response displacement, resisting force, absolute acceleration, and input ground motion parameters including peak ground acceleration (PGA) are compiled. Exploratory analysis of maximum absolute acceleration in relation to other structural response parameters suggests that the square root of the log of the absolute acceleration divided by resisting force has a piecewise linear relationship with log of maximum response displacement as shown in Figure 3a. Subsequently, we explore a better fitting model by adding an input ground motion parameter for a predictor variable, and conclude that inclusion of the log of PGA makes the residuals and the fitted values uncorrelated as shown in Figure 3b. Equation 2 summarises the regression equation developed from the process.

$$\sqrt{\ln\left(\frac{S_{aa}}{S_{a}}\right)} = c + S_{aa,sd} + S_{aa,PGA} + \varepsilon$$

$$(2)$$
where $S_{aa,sd} = \begin{cases} a_1 \times bp_{sd} + (a_1 + a_2) \times (\ln(S_d) - bp_{sd}), \ln(S_d) \ge bp_{sd} \\ a_1 \times bp_{sd} + (a_1 + a_2) \times (\ln(S_d) - bp_{sd}), \ln(S_d) \ge bp_{sd} \end{cases}$

$$S_{aa,PGA} = \begin{cases} b_1 \times bp_{PGA} + (b_1 + b_2) \times (\ln(PGA) - bp_{PGA}), \ln(PGA) \le bp_{PGA} \\ b_1 \times bp_{PGA} + (b_1 + b_2) \times (\ln(PGA) - bp_{PGA}), \ln(PGA) \ge bp_{PGA} \end{cases}$$

where a_1 , a_2 , b_1 , b_2 , c, bp_{sd} , and bp_{PGA} are regression coefficients; S_{aa} is the maximum absolute acceleration; $S_{aa,sd}$ and $S_{aa,PGA}$ are piecewise linear functions of log of maximum response displacement, S_d , and log of PGA of input ground motion, respectively; S_a is the maximum resisting force from the time history analysis or the spectral acceleration of the performance point based on the CSM, and ε is error.

The regression coefficients are determined by segmented regression analysis using the *segmented* package (Muggeo, 2008) in R (R Core Team, 2015). Table 1 summarises the regression coefficients for S3-IND2, Pre-code. Note that all regression coefficients are computed with response displacement in mm, PGA in g, and spectral acceleration in g, respectively.



Figure 3. a) $\sqrt{\ln(S_{aa}/S_a)}$ versus $\ln S_a$ from time history analysis of S3-IND2, Pre-code, and b) Standarised residual versus predicted value of $\sqrt{\ln(S_{aa}/S_a)}$ from regression analysis for S3-IND2, Pre-code

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Coefficient	Estimated value	Standard error	t value	p value
С	0.229	0.058	3.916	< 0.001
<i>a</i> ₁	0.045	0.015	3.034	0.003
<i>a</i> ₂	0.080	0.015	5.298	NA
bp _{sd}	2.766	0.104	NA	NA
b_1	0.062	0.013	4.646	< 0.001
<i>b</i> ₂	0.110	0.014	7.777	NA
bp _{PGA}	-1.019	0.075	NA	NA
N=656	R ² =0.972	$R_a^2 = 0.972$	<i>σ</i> =0.046	DOF= 649

Table 1. Regression analysis result for S3-IND2, Pre-code

Note: *N* is the total number of data, R^2 is the coefficient of determination, R_a^2 is the adjusted R^2 , $\hat{\sigma}$ is the standard deviation of the regression residuals, and DOF is the degree of freedom.

Using the regression equation for maximum absolute acceleration, the new fragility for the NSA components and the resulting vulnerability curves for S3-IND2, Pre-code are computed and compared with the current curves as shown in Figure 4. It is cleary shown that the discrepancy between the two curves becomes larger with increasing ground shaking intensity.



Figure 4. a) Comparison of the NSA component fragility, and b) vulnerability curves of S3-IND2, Precode

4 EFFECT OF THE NEW MODELS TO PORTFOLIO RISK

To illustrate the effect of the new vulnerability models, a portfolio of buildings in Newcastle is created with a modification of the portfolio used for the Newcastle earthquake risk assessment (Fulford et al., 2002). The number of buildings in the Newcastle portfolio is 6305, and it is reduced to 1066 buildings representing two building types only for illustration purposes. The modified building portfolio consists of 778 of W1 and 288 of S3 buildings. The occupancy types of W1 and S3 are set to be HAZUS occupancy type RES1 (Single Family Dwelling Detached House) and IND2 (Light Industrial Factory), respectively. The repair cost ratios in percentage of building replacement cost associated with the structural, NSD, and NSA components are 23.4, 50.0, 26.6 for RES1 while 15.7, 11.8, and 72.5 for IND2.

The same earthquake source and site response models developed in the Newcastle earthquake risk assessment (Dhu et al., 2002) are used to generate a large number of ground motion fields. For each generated ground motion field, economic loss of each building is computed via the aforementioned CSM-based approach, and the loss ratio is computed as a fraction relative to the sum of the replacement cost of the buildings. As shown in Figure 5, larger loss ratios are estimated with the new NSA component fragility and the resulting vulnerability models than the current models. The tendency is more noticeable for S3-IND2 whose repair cost ratio for the NSA component is larger than W1-RES1.



Figure 5. a) Comparison of loss exceedance curves for the portfolio, and b) Comparison of loss exceedanc curve by building type between the current and the new vulnerability models.

5 CONCLUSIONS

A limitation of the current CSM-based approach in computing NSA component fragility is identified and an approach for correcting the limitation is proposed through regression analysis on absolute acceleration obtained from time history analysis of a SDOF system with an elliptical hysteresis. The new NSA component fragility is compared with the current one, and the effect of the new model is highlighted in probabilistic risk analysis for a portfolio of buildings in Newcastle. The current approach results in lower estimates of loss than our newly proposed approach, and we believe our new approach produces more accurate loss estimates due to its more realistic treatment of NSA components. The difference between the two approaches is especially significant for occupancy types for which the repair cost ratio for the NSA component is larger than other components such as IND2. The new approach can be applied for other building types and the effect of various hysteresis models on the maximum absolute acceleration needs to be investigated.

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