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Effect of construction quality variability on seismic fragility of reinforced concrete building

P. Rajeev

Monash University, Melbourne, VIC, Australia.

S. Tesfamariam

The University of British Columbia, Kelowna, BC, Canada.

ABSTRACT: This paper discusses effect of construction defects on probabilistic seismic demand model (PSDM) of reinforced concrete (RC) frames. A six-storey three-bay moment resisting RC frame is designed to a 1984 Canadian Concrete Design Code. The RC frame is further modified to investigate variability of construction quality (CQ) on the PSDM. Three levels of CQ are considered, *poor*, *average*, and *good*. Forty five ground motion records were used to study the ground motion variability. The numerical model of the frame was developed in OpenSees and nonlinear dynamic analyses were performed, and the maximum interstorey drift is obtained as a response parameter for all simulations. The PSDM parameters are calculated using "cloud analysis" for all combinations of construction quality. The variation in the PSDM parameters is studied. Finally, the effects of CQ on the seismic fragilities are discussed.

1 INTRODUCTION

Design and construction practices have significantly improved over the years, yet earthquakes continue to cause severe damage and losses (Tesfamariam and Saatcioglu, 2008). Recent earthquakes, Haiti and Chile in 2010, Italy in 2009, El Salvador and India in 2001, and Turkey and Taiwan in 1999, for example, highlight the seismic vulnerability of existing buildings and resulting damage. The reported causes of building damage entail poor quality materials, inadequate reinforcement detailing and absence of capacity design principles (e.g., Pampanin *et al.*, 2002; Elnashai, 2000; Sezen *et al.*, 1999). However, no clear trend can be observed towards the correction of the inadequate construction practices that have been time and again observed from various earthquakes (Meli and Alcocer, 2004).

Though quality of material and construction are reported cause of damage, there are very few studies undertaken on this subject. Pampanin *et al.* (2002) tested 2/3-scaled beam-column sub-assemblages, with structural deficiencies prevalent in 1950's and 1970's Italian construction practice. Lu *et al.* (2001) experimentally estimated the seismic capacity of a two-storey reinforced concrete (RC) frame with insufficient confinement in the critical zones of the columns. Dimova and Negro (2005, 2006) have experimentally and analytically quantified the effect of construction quality defects, resulted from deficiencies in arrangement of the reinforcement, on the seismic performance of a cast-in-situ one-storey industrial reinforced concrete frame designed according to Eurocodes.

In this study, the effects of construction quality (CQ) on probabilistic seismic demand model (PSDM) and seismic fragility of RC frames is analytically investigated. The variation in material and structural detailing are considered as the cause of CQ uncertainty. Interaction of different material and structural detailing on the response of the frame is also presented. Since the damage to buildings can be related to the interstorey drift (e.g., FEMA 356 (FEMA 2000)), the seismic demand model and fragility are derived in terms of interstorey drift.

2 BUILDING DESIGN CONSIDERATION

A six-storey three-bay moment resisting RC frame is designed to a 1984 Canadian Concrete Design Code. The building is for office use and is located in Vancouver, Canada, which is considered to be high seismic hazard zone (NBCC 2005). The plan of the building is 24 m x 42 m, and the storey heights are 3.65 m. The distance between the longitudinal frames is 6 m. The lateral load resisting system consists of moment-resisting RC frames both in longitudinal and the transverse directions. Secondary beams between the longitudinal frames are used at the floor levels in order to reduce the depth of the floor slabs. The floor system consists of a one-way slab spanning in the transverse direction. In this study, only the interior transverse frame of the building is considered (Figure 1).

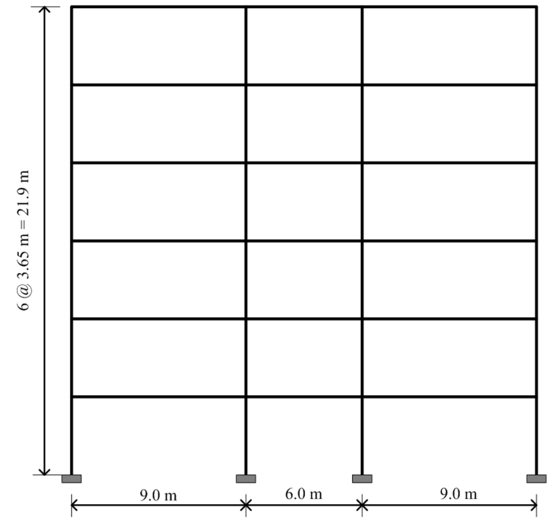


Figure 1. Elevation of the six-storey structure located in Vancouver

3 FINITE ELEMENT MODELLING OF RC FRAME

Finite element analysis of the frame was performed using OpenSees finite element analysis package (McKenna *et al.* 2007). Columns are modelled with distributed plasticity nonlinear beam-column elements while the beams are modelled with beam with hinge elements. P-Δ effect is considered and tangent-stiffness proportional damping has been used, calibrated to yield a 5% equivalent viscous damping ratio on the first elastic mode. Concrete behaviour is modelled by a uniaxial Kent-Scott-Park model (*Concrete01*) with degrading, linear, unloading/reloading stiffness, without consideration of tensile strength. For the confined concrete, the strength and strain values have been increased according to the formulae developed by Mander *et al.* (1988). Steel behaviour is represented by a uniaxial Giuffre-Menegotto-Pinto model (*Steel02*).

4 VARIABILITY IN MATERIAL PROPERTIES AND STRUCTURAL DETAILING

The CQ is quantified by varying the material and structural detailing. The corresponding variabilities are categorized into three CQ levels {poor, average, good} as summarized in Table 1. The material uncertainty parameters considered are compressive strength of concrete ($f_c^{'}$), yield strength of reinforcing steel (f_y), and hardening ratio of steel (b_h). The structural detailing uncertainties considered are tie spacing at the column (s_c) and reinforcement ratio at the column (ρ). The material uncertainty is due to the different construction phases, while the structural detailing uncertainty is due to the different design methods and factor of safety considered in the design and construction phase.

Table 1. Material and structural detailing uncertainty

Material uncertainty	**Unit**	**Poor**	**Average**	**Good**
Compressive strength of concrete ($f_c^{'}$)	MPa	25	35	45
Yield strength of reinforcing steel (f_y)	MPa	290	345	400
Hardening ratio of steel (b_h)	%	1.25	2.00	2.75
Structural detailing uncertainty	**Unit**	**Poor**	**Average**	**Good**
Tie spacing at the column (s_c)	mm	300	200	100
Reinforcement ratio at the column (ρ)	%	0.8	1.0	1.6

5 EARTHQUAKE GROUND MOTIONS

In order to perform time-history analyses, an ensemble of 45 ground motion records (three groups of 15 records) are selected[1] (Naumoski *et al*. 2006). Each group matches different acceleration to velocity (*A/V*) ratios, i.e. accelerograms with high *A/V* ratios ($A/V > 1.2$), intermediate *A/V* ratios ($0.8 < A/V < 1.2$), and low *A/V* ratios ($A/V < 0.8$), where *A* is in g, and *V* is in m/s. The selected accelerograms are recorded on rock or stiff soil sites, including large and distant, large and close, moderate and close as well as intermediate earthquake records. The magnitude (M_w) range from 5.25 to 7.81 and source-to-site distances (r) range is $4 < r < 379$ km. These records are representative distances and expected magnitude ranges of Canadian earthquakes. Figure 2 shows the target uniform hazard spectrum of 10%, 5%, and 2% in 50 years return period for Vancouver, and the elastic 5% damped response spectrum of each 15 records scaled to match the average elastic spectrum T_1. The fundamental first mode period (T_1) of the base structure is 1.83 s, where the material and structural detailing have values of *average* (Table 1).

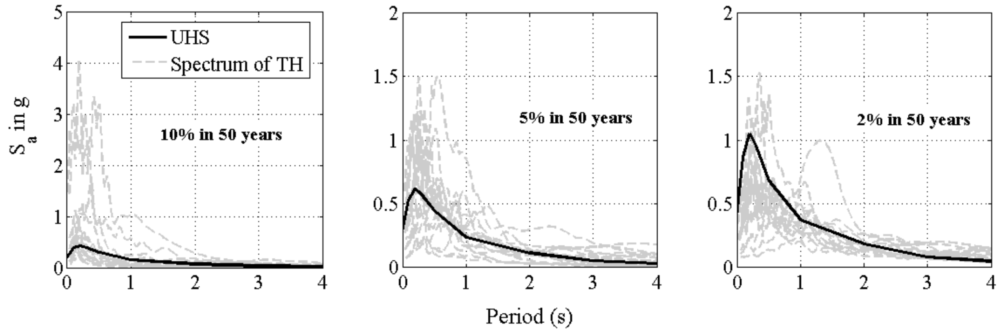


Figure 2. Target and average demand spectra for Vancouver, Canada

6 PROBABILISTIC SEISMIC DEMAND MODEL AND FRAGILITY

A probability distribution for the demand conditioned on the intensity measure (*IM*) is known as a probabilistic seismic demand model (PSDM). The demand on the structure is quantified using some chosen metric(s) (e.g. inter storey drift, ductility). Cornell *et al*. (2002) suggested that the estimate for the median demand ($\hat{D}$) can be represented by the following power model:

$$\hat{D} = a \cdot IM^{b} \tag{1}$$

where *IM* is the seismic intensity measure of choice; both *a* and *b* are regression coefficients. In this study, spectral acceleration at the first mode period of the structure $S_a(T_1)$ is selected as the *IM* and the interstorey drift ($\theta_{\max}$) is selected as the demand parameter.

The nonlinear dynamic analyses can be used to quantify the PSDM parameters. One procedure, known as “Cloud Analysis” (Jalayer *et al.,* 2007), is a convenient choice (though not the most accurate). An advantage of this method is that it is based on the ground motions as they are recorded and does not require scaling. The procedure consists of applying a suite of ground motion records (in the order of 10-30 records) to the structure and calculates the demand *D*. Then, by performing a simple linear regression of the logarithm of *D* against the logarithm of *IM*, one can obtain the PSDM parameters *a* and *b*.

Furthermore, the distribution of the demand about its median is often assumed to follow a two parameter lognormal probability distribution. Thus, the dispersion ($\beta_{D|IM}$) of the demand about its median can be computed and is conditioned upon the *IM.* The dispersion is assumed be constant for the range of *IM* values interested.

The fragility is simply the probability that the seismic demand (*D*) placed on the structure is greater than the capacity (*C*) of the structure. This probability statement is conditioned on a chosen *IM*, which

[1] Source: http://www.caee.uottawa.ca/Publications/Earthquake%20records/Earthquake%20Records.htm

represents the level of seismic loading. The generic representation of this conditional probability is:

$$Fragility = P(D > C \mid IM) \tag{2}$$

The fragility function (Equation 2) can be evaluated by convolving PSDM with a distribution of the capacity. As explained above, due to the lognormal distribution assumption of the demand at each level of *IM* and capacity, the conditional probability can be expressed as:

$$P(D > C \mid IM) = 1 - \Phi\left(\frac{\ln(\hat{C}) - \ln(a \cdot IM^{b})}{\beta_{D \mid IM}}\right) \tag{3}$$

where $\hat{C}$ is the median structural capacity, associated with the limit state.

6.1 *Impact of Construction Quality in PSDM*

To study impact of CQ on the PSDM parameters, the RC frame's material properties and detailing are varied considering the variability summarized in Table 1. Since the number of uncertain parameter is five, where each has three levels, the number of sample RC models is 243 (3^5). The maximum interstorey drift $\theta_{\max}$ is computed for each sample structure by performing the nonlinear time history analysis with the 45 ground motion records. Thus, the total number of time history analysis carried out is 10935 (45*243). The seismic demand model is developed for each sample structure (Equation 1). The probabilistic seismic demand model and model parameters (a, b and $\beta_{\theta_{max} \mid S_a(T_1)}$) are shown in Figure 3 for the base structure. The histogram of the distribution of PSDM parameters is shown in Figure 4. Table 2 summarizes the statistical property of PSDM parameters. It can be observed that the a and $\beta_{\theta_{max} \mid S_a(T_1)}$ show more than 10% coefficient of variation (CoV).

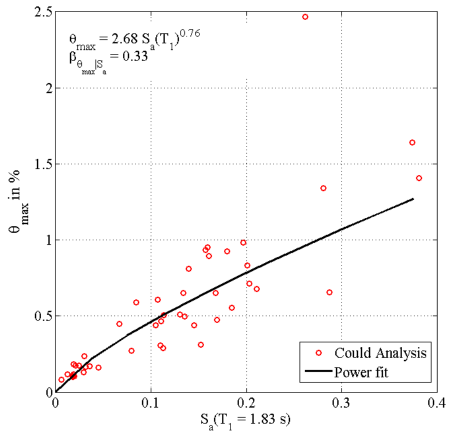


Figure 3. Probabilistic seismic demand model of base structure

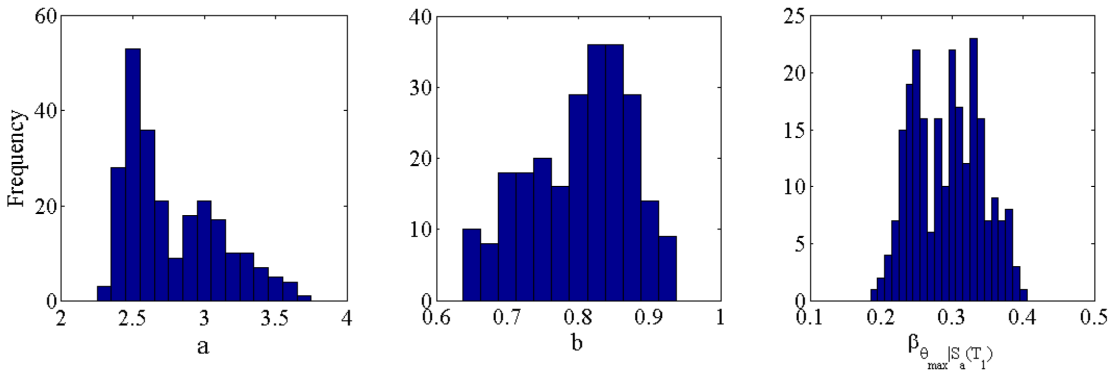


Figure 4. Histogram of PSDM parameter

Table 2. Statistical property of PSDM parameters

Parameter	Median	CoV (%)	Range
a	2.66	12.24	[2.32, 3.71]
b	0.82	8.71	[0.65, 0.93]
$\beta_{\theta_{max}\|S_a(T_1)}$	0.30	16.01	[0.19, 0.40]

Figure 5 illustrates sensitivity of PSDM parameters on the variability of CQ. The sensitivity analysis is carried out by changing one parameter at a time, where for each level of the parameter (Table 1), corresponding the *minimum*, *average* and *maximum* values of PSDM parameters are computed.

Parameter *a* shows (Figure 5a):

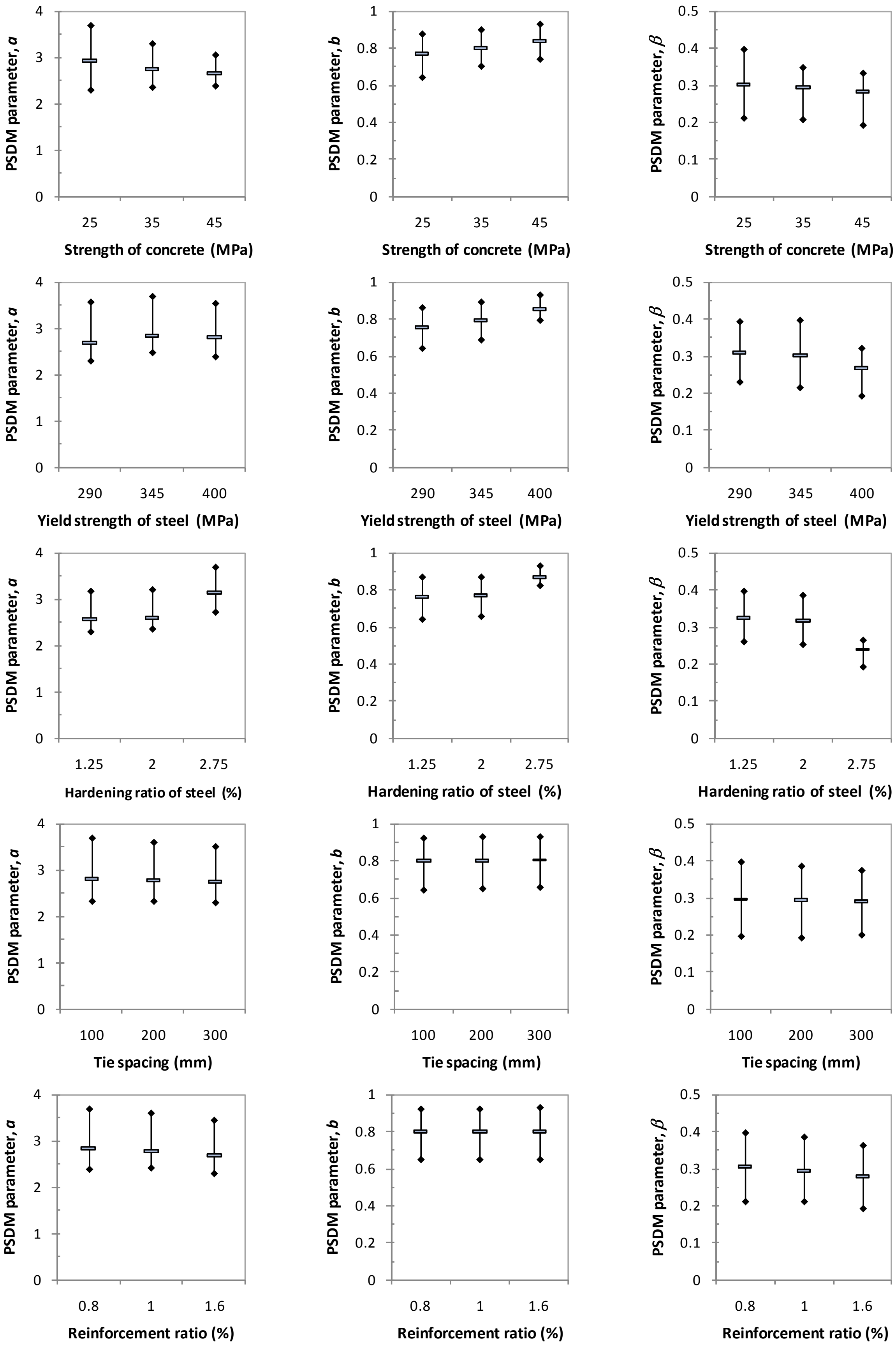
1. decreasing trend with increase in $f_c^{'}$ and ρ,
2. increasing from poor (290 MPa) to average (345 MPa) then decreasing with increase in f_y, and
3. increasing trend with increase in b_h and s_c.

Parameter *b* shows (Figure 5b):

1. increasing trend with increasing $f_c^{'}$, f_y, and b_h,
2. decreasing trend with increase in s_c, and
3. increasing from poor to average and keep the same value from average to good with ρ.

Parameter $\beta_{\theta_{max}|S_a(T_1)}$ shows (Figure 5c):

1. decreasing trend with increasing $f_c^{'}$, f_y, b_h and ρ, and
2. increasing trend with increase in s_c.



(a) (b) (c)

Figure 5. Sensitivity of PSDM parameter to variablity in CQ: (a) *a*, (b) *b*, and (c) $\boldsymbol{\beta}_{\theta_{max}|S_a(T_1)}$

6.2 *Impact of Construction Quality in Seismic fragilities*

To generate the fragilities, the capacities are defined by the maximum interstorey drift that correspond to three performance levels, immediate occupancy (IO), life safety (LS), and collapse prevention (CP). Table 3 presents the medians of θ_{max} associated with these limit states according to FEMA 356 (FEMA 2000).

Table 3. Parameters used for estimating capacity

Parameter	Limit state	Interstorey drift limit (%)
$\hat{C}$ (%)	IO	1
	LS	2
	CP	4

Results of the fragility curves are depicted in Figure 6. Figure 6 shows the 95% confidence bounds on all sampled frame model fragilities, median fragility and average CQ model frame fragilities. Table 4 gives the median and corresponding fractile lognormal distribution parameters ($\hat{S}_C(g)$ and logarithmic standard deviation ξ).

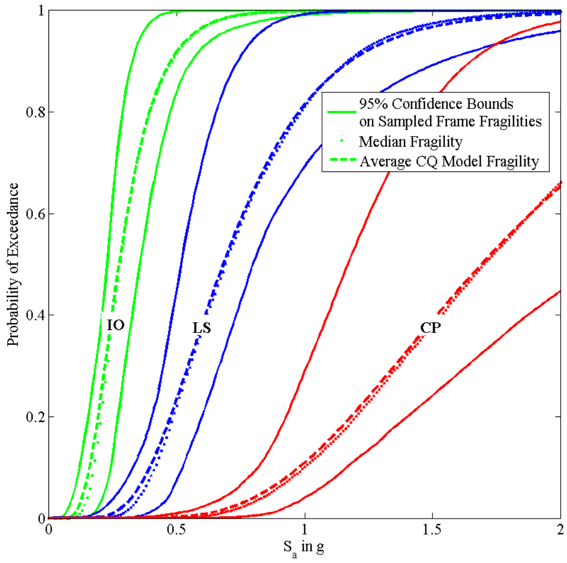


Figure 6. The 95% confidence bounds on all sampled frame model fragilities, median fragility and average CQ model frame fragility

Table 4. Fragility parameters

Percentile	$\hat{S}_C(g)$			ξ		
	IO	LS	CP	IO	LS	CP
2.5	0.44	0.89	2.07	0.40	0.32	0.33
50	0.30	0.71	1.68	0.38	0.37	0.37
97.5	0.18	0.43	1.13	0.36	0.34	0.36
Average CQ model	0.28	0.68	1.69	0.43	0.43	0.42

Figure 6 and Table 4 show that median fragility of sampled frame and the average CQ model fragility are bounded between the 95% confidence bound for all limit states. The median spectral capacity computed using the sampled structure's fragility curve is very close to the average CQ structural model capacity. It can also be noted that uncertainty in the fragility function increases across the limit states, from IO to CP. The uncertainty in the fragility of CP is significantly less in the lower probabilities that at the higher probabilities. The median fragility and average CQ model fragility do not show much variation, however the median fragility is below the average CQ model fragility at low cumulative probabilities and above at high cumulative probabilities.

7 DISCUSSION AND CONCLUSIONS

A six storey reinforced concrete frame designed for Vancouver, Canada was used in this study. Three group of 15 ground motion records representative of three levels of *A/V* range is used. Each group is scaled to match the target spectrum of 10%, 5% and 2% exceedance in 50 years in the Vancouver region. The material and structural detailing uncertainty is considered and each uncertain parameter has three levels, such as {poor, average and good}. The different combinations of the uncertain parameters are used to generate different structures. The maximum interstorey drift θ_{max} obtained from nonlinear time history analysis of the frames was considered as a response parameter, and the spectral acceleration at the fundamental structural period $S_a(T_1)$ was considered as an intensity measure to develop the PSDM. The PSDM is convoluted with capacity of the structure to generate the fragility curves.

The results showed that the PSDM parameters *a*, *b* and $\beta_{\theta_{max}|S_a(T_1)}$ has considerable dependence in CQ and the CoV is nearly or above 10% for all three parameters. The fragility curves obtained for the frame show larger variability, which depends on the limit state. The median spectral capacity computed using the sampled structure's fragility curve is very close to the average CQ structural model capacity. Preliminary analysis showed that the PSDM and fragility are quite sensitive to CQ. This subject, however, is still under investigation by this authors.

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