Seismic Risk Assessment of Mega-thrust M_w9-class Subduction Earthquakes and Aftershocks in Victoria, British Columbia, Canada Using Multi-variate Seismic Demand Models

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ABSTRACT: This study extends current seismic demand estimation methods based on incremental dynamic analysis by characterizing dependence among different engineering demand parameters using copulas explicitly. The developed method is applied to a 4-story non-ductile reinforced concrete frame in Victoria, British Columbia, Canada. The developed multi-variate seismic demand models are integrated with a story-based damage-loss model to assess the seismic risk due to different earthquake loss generation modes (i.e. non-collapse repairs, collapse, and demolition). Results obtained from this study indicate that the effects of multi-variate seismic demand modeling on the expected seismic loss ratios are significant. The critical information is the limit state threshold for demolition. In addition, consideration of a realistic dependence structure of maximum and residual inter-story drift ratios can be important for seismic loss estimation as well as for multi-criteria seismic performance evaluation.

1. INTRODUCTION

Victoria is the capital city of British Columbia (BC), Canada, on Vancouver Island, facing the Strait of Juan de Fuca. Previous regional seismic risk assessment exercises carried out by Onur et al. (2005) and AIR Worldwide (2013) indicated that seismic risk to vulnerable parts of older constructions in Victoria is high. In this paper, with consideration of M_w9-class subduction earthquakes and aftershocks, seismic risk assessment is carried out for a 4-story non-ductile reinforced concrete (RC) building.

An accurate assessment of potential impact of future destructive earthquakes is essential for effective disaster risk reduction. The performance-based earthquake engineering (PBEE) methodology has been developed to assess seismic vulnerability of structures probabilistically (Cornell et al. 2002). The PBEE methodology can be expressed based on total probability theorem:

$$\nu(DV) = \iint G(DV \mid EDP) \left| dG(EDP \mid IM) \right| \left| d\lambda(IM) \right| \quad (1)$$

λ(IM) is the mean annual rate of exceeding a given seismic intensity measure (IM) level and is obtained from probabilistic seismic hazard analysis (PSHA). The structural analysis develops a probabilistic relationship between IM and engineering demand parameter (EDP), which is denoted by the complementary cumulative probability distribution function *G*(EDP|IM). Typical EDP parameters include the maximum and residual inter-story drift ratios (denoted by MaxISDR and ResISDR, respectively) and peak floor acceleration (PFA) for structural and non-structural components. The damage-loss analysis relates EDP to seismic performance metrics, parameterized with decision variables (DVs), such as repair/reconstruction costs, downtime, and casualties. Equation (1) is formulated based on so-called EDP-DV functions (i.e. *G*(DV|IM)) by Ramirez and Miranda (2009).

Parameterization of earthquake damage and loss generation processes has major influence on the computation and modeling of EDP. The EDP is either uni-variate or multi-variate. The multi-variate case is often implemented using fragility models for different types of damage sensitivity. However, fragility curves for different EDPs are evaluated separately and thus dependence of EDP variables for a given IM is not explicitly taken into account. Goda (2010) and Uma et al. (2010) investigated the joint probabilistic modeling of the two inter-related parameters using inelastic single-degree-of-freedom systems. However, rigorous evaluation of joint probability distributions of MaxISDR and ResISDR for realistic multi-degree-of-freedom systems has not been carried out. Therefore, this study investigates the joint probabilistic modeling of multiple EDPs by conducting statistical characterizations of marginal probability distributions for MaxISDR, ResISDR, and PFA and copula (dependence) models between MaxISDR and ResISDR. A copula technique offers a flexible way of describing nonlinear dependence among multi-variate data in isolation from their marginal distributions, and serves as a powerful tool for modeling nonlinearly-interrelated multi-variate data (McNeil et al. 2005).

The novel contributions of this study are: (i) copula-based multi-variate modeling of EDPs is conducted for a realistic structural model, and (ii) the impact of multi-variate seismic demand modeling on seismic risk assessment is investigated. The former essentially extends the current incremental dynamic analysis (IDA)-based uni-variate seismic demand modeling approaches.

2. DEPENDENCE MODELING USING COPULAS

Consider the joint probability distribution of two random variables X_1 and X_2, $H(x_1,x_2) = P[X_1 \leq x_1, X_2 \leq x_2]$. Continuous marginal probability distributions are denoted by $F_1(x_1)$ (= u_1) and $F_2(x_2)$ (= u_2), respectively. u_1 and u_2 represent a sample of a standard uniform random variable U_1 and U_2, respectively, and $P[\bullet]$ represents the probability. Sklar's theorem dictates that a relationship among $H(x_1,x_2)$, $F_1(x_1)$, and $F_2(x_2)$ can be established by using the copula function $C(u_1,u_2)$ (McNeil et al. 2005):

$$H(x_1, x_2) = C(F_1(x_1), F_2(x_2)) = C(u_1, u_2) \quad (2)$$

In other words, the joint probability distribution of the two random variables can be characterized by a copula function in terms of their marginal probability distributions. An important implication of this theorem is that marginal modeling and dependence modeling can be carried out separately.

One of the copula functions that are applicable to the joint probabilistic modeling of MaxISDR and ResISDR is the asymmetrical Gumbel copula (Goda 2010). The bi-variate asymmetrical Gumbel copula $C_{\theta,w1,w2}(u_1,u_2)$ is given by (McNeil et al. 2005):

$$C_{\theta,w_1,w_2}(u_1,u_2) = u_1^{1-w_1} u_2^{1-w_2} C_\theta(u_1^{w_1}, u_2^{w_2}) \quad (3)$$

and

$$C_\theta(u_1,u_2) = \exp\left(-[(-\ln u_1)^\theta + (-\ln u_2)^\theta]^{1/\theta}\right) \quad (4)$$

where w_1 and w_2 are the weight parameters and range from 0.0 to 1.0. When $w_1 = w_2 = 1.0$, $C_{\theta,w1,w2}(u_1,u_2)$ equals the Gumbel copula $C_\theta(u_1,u_2)$, while when $w_1 = w_2 = 0.0$, $C_{\theta,w1,w2}(u_1,u_2)$ equals the independence copula $C(u_1,u_2) = u_1u_2$. Various kinds of asymmetrical copulas can be constructed by varying values of w_1 and w_2. The parameters of asymmetrical Archimedean copulas can be obtained based on the maximum likelihood method.

3. MULTI-VARIATE SEISMIC RISK ASSESSMENT WITH CONSIDERATION OF SEISMIC DEMAND DEPENDENCE

3.1. Building design consideration

A 4-story office building (Figure 1a), with an RC space frame lateral resisting system (Liel and Deierlein 2008), has a floor plan measuring 125 ft by 175 ft and columns spaced at 25 ft. The total height of the structure is 54 ft, with ground and higher floor levels story heights being 15 ft and 13 ft, respectively. The 1967 Uniform Building Code (UBC) seismic provisions are applied. The structure is designed as a space frame, such that all columns and beams are part of the lateral resisting system. All beam and column elements have the same amount of over-strength; each element is 15% stronger than the code-minimum design level. The design is governed by strength and stiffness requirements, as the 1967 UBC had few requirements for special seismic design or ductile detailing.

The non-ductile structure is modeled in OpenSees using a lumped plasticity approach. The lumped plasticity element models used to simulate plastic hinges in beam-column elements utilize a nonlinear spring model (Figure 1b). This model is capable of capturing important modes of deterioration that lead to side-sway collapse of RC frames. Modal analysis of the finite-element model indicates that the first three modal periods are 1.92, 0.55, and 0.27 s, respectively.

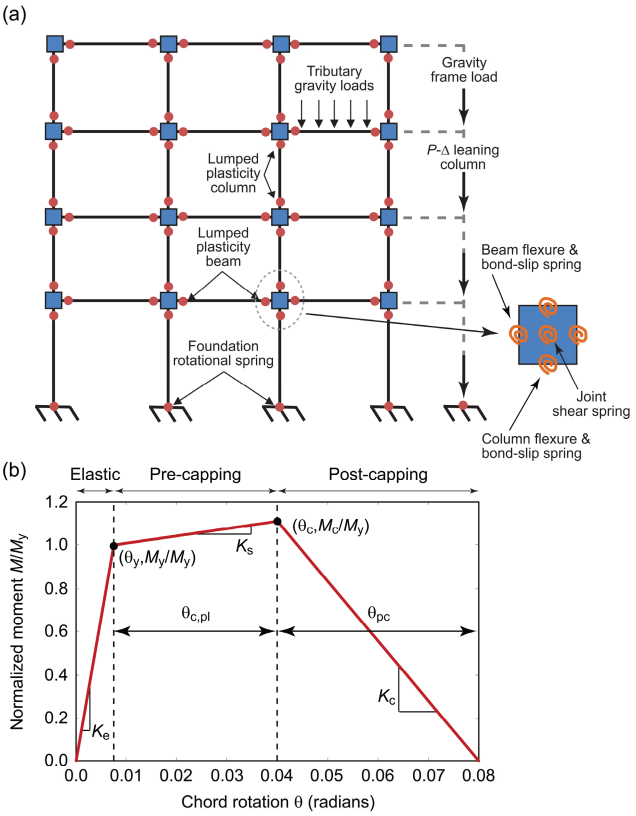

Figure 1: (a) Nonlinear finite-element model of a 4-story non-ductile RC frame and (b) backbone curve of an element model used for beam-column elements.

3.2. Seismic hazard and ground motion selection for British Columbia, Canada

Three potential sources of damaging earthquakes are prevalent in BC: shallow crustal earthquakes, deep inslab earthquakes, and off-shore mega-thrust interface earthquakes from the Cascadia subduction zone (Hyndman and Rogers 2010). The expected magnitude of the Cascadia events is in the range of M_w8 to M_w9; its mean recurrence period ranges from 500 to 600 years and the last event occurred in 1700.

The input ground motion records for use in incremental dynamic analysis (IDA; see Section 3.3) need to be selected carefully, because record scaling may induce bias in calculated structural responses. For this purpose, a new ground motion record database has been compiled by incorporating recent recordings from Japanese K-NET, KiK-net, and SK-net (including the 2011 M_w9 Tohoku earthquake records), as well as worldwide crustal earthquake records (PEER-NGA database). The Tohoku dataset is particularly relevant to the Cascadia event, because of anticipated macro-level similarity between these two mega-thrust subduction events. The combined database is comprised of 606 mainshock-aftershock (MSAS) record sequences. An innovative aspect of the database is that all time-history data are associated with actual MSAS sequences. This is advantageous in evaluating the seismic loss due to both mainshocks and aftershocks (Tesfamariam and Goda 2015).

To avoid bias due to excessive record scaling in assessing seismic performance of a structure, a multiple conditional mean spectra (CMS) method is implemented by reflecting regional seismic hazard characteristics in BC (Goda and Atkinson 2011). The dominant earthquake scenarios that are necessary to define multiple target spectra (i.e. CMS) for three earthquake types (crustal/interface/inslab) are obtained from PSHA for Victoria (Figure 2a). The return period of 2500 years is focused upon. It is noteworthy that major contributions to overall hazard in Victoria are originated from the

Cascadia subduction zone (Figure 2b). This is an important consideration in selecting records for seismic performance evaluation of structures in Victoria.

Using the constructed ground motion database, 50 records (two horizontal components per record; i.e. 100 time-history data) are selected based on the multiple CMS method. The response spectra of the selected records match the target CMS over the vibration period range between 0.3 and 3.0 s. The number of records for each earthquake type, out of 50 records, is determined based on its relative contribution to seismic hazard using PSHA results. Specifically, for the return period of 2500 years, the number of records for crustal, interface, and inslab events is 13, 25, and 12, respectively.

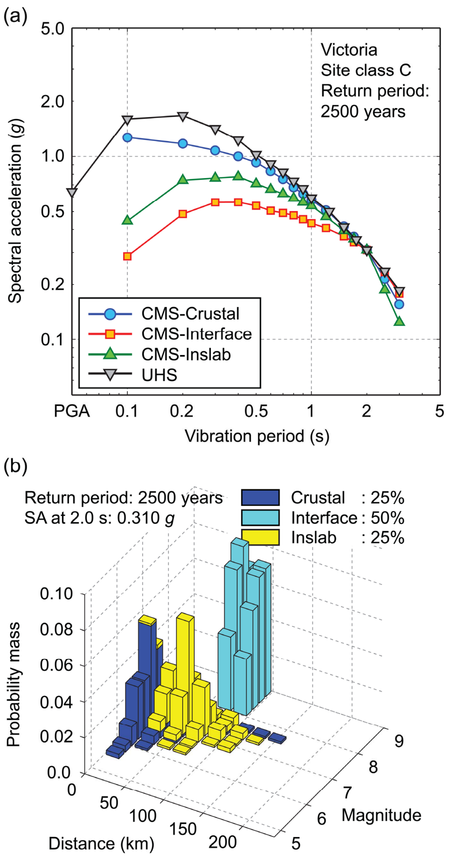

Figure 2: Comparison of UHS and CMS (a) and seismic deaggregation (b) for Victoria (site class C) at the return period of 2500 years.

3.3. Incremental dynamic analysis

IDA is carried out for the 4-story non-ductile RC frame using a set of 50 MSAS sequences as well as a set of 50 MS records to evaluate the effect of aftershocks. The IM is specified in terms of spectral acceleration at 2.0 s and ranges from 0.05 to 0.7 *g*. In general, numerical instability is encountered when the ISDR of the frame exceeds 0.10, and it is treated as a *collapse* indicator in this study.

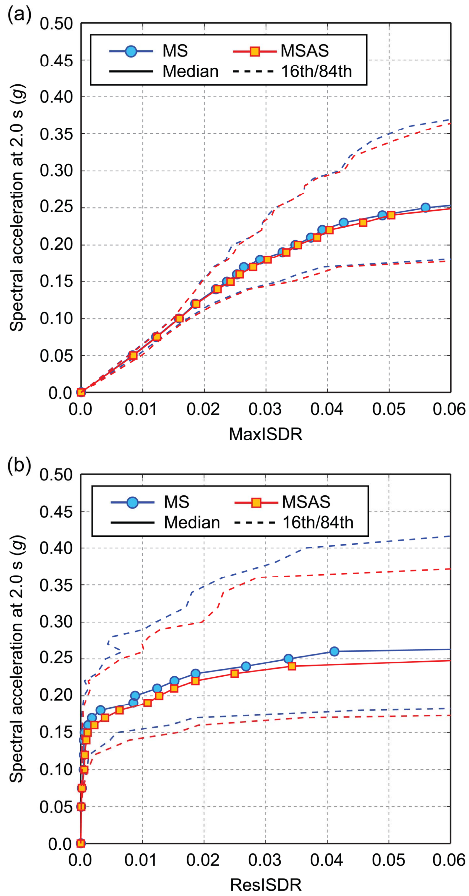

Figure 3: IDA results for MaxISDR (a) and ResISDR (b) by considering MS records only and MSAS sequences.

The main IDA results for both MS records and MSAS sequences are shown in Figure 3 for MaxISDR and ResISDR (note: results for PFA are omitted for brevity). To present the uncertainty of the IDA results succinctly, median curves as well as 16th-84th curves are included in the figure. The results shown in Figure 3 suggest that the overall characteristics of the IDA curves for MaxISDR and ResISDR are different; the former increases gradually with the seismic intensity level, while the latter increases rapidly when the seismic intensity level reaches about 0.15 *g* (at which the corresponding median MaxISDR is about 0.025). The uncertainty of ResISDR is much greater than that of MaxISDR. Inspection of Figure 3 indicates that the aftershock effects on MaxISDR are noticeable at high seismic intensity levels only, whereas those on ResISDR can be recognized more easily.

3.4. Multi-variate modeling of engineering demand parameters

Preliminary data analysis is carried out to identify the key EDPs in developing multi-variate seismic demand prediction models. From the analysis, large responses are observed at the ground floor, influenced by potential soft story failure mechanism. Therefore, MaxISDR, ResISDR, and PFA at the ground floor are focused upon for probabilistic modeling. The preliminary analysis also indicates that the correlation between MaxISDR and ResISDR is moderate, while that between MaxISDR/ResISDR and PFA is small. Based on this, MaxISDR and ResISDR are treated as dependent variables, whereas PFA is considered to be independent from the other two.

Next, suitable marginal probability distribution types for MaxISDR, ResISDR, and PFA are examined. The suitable probability distributions for individual EDP parameters are determined by comparing the statistical metrics, such as Akaike Information Criterion (AIC), for different statistical distributions. For MaxISDR and PFA, the Frechet distribution is adequate (lognormal distribution is also applicable). For ResISDR, the generalized Pareto distribution is suitable to capture the heavy right-tail characteristics. The non-normal characteristics of ResISDR suggest that the use of copulas, rather than conventional multi-(log)normal distributions, is advantageous as marginal and dependence modeling can be performed separately.

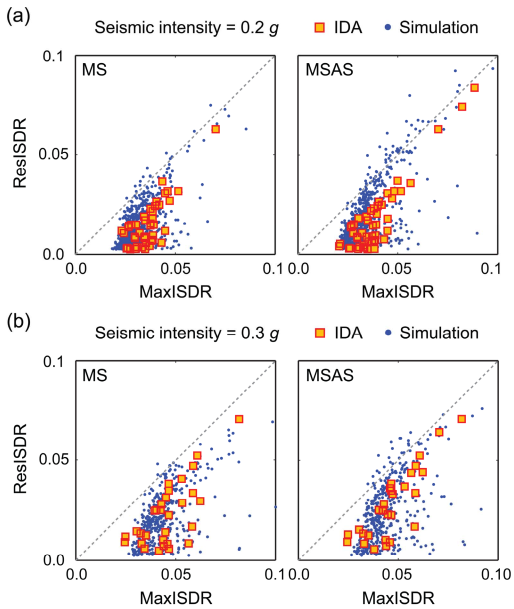

Figure 4: MaxISDR-ResISDR scatter plots based on the IDA results and simulated samples at seismic intensity levels of 0.2 g (a) and 0.3 g (b).

Subsequently, dependence modeling of MaxISDR and ResISDR is carried out using copulas. The modeling results highlight that the asymmetrical Gumbel copula is suitable for the majority of the cases examined in this study (based on the AIC). Accuracy of the joint probabilistic modeling of MaxISDR and ResISDR at the ground floor can be examined visually by comparing simulated samples of these variables with the IDA results at various seismic intensity levels. The results, for 1000 simulated samples, are shown in Figure 4 for MS records and MSAS sequences at the target intensity levels of 0.2 and 0.3 *g*. Overall, it can be concluded that the joint probability distribution modeling of MaxISDR and ResISDR is satisfactory. For some cases, there are *unrealistic* pairs (i.e. ResISDR exceeds MaxISDR); the boundary between realistic and unrealistic cases is shown by a dotted line. Typically, 1 to 4% of the samples may fall into unrealistic cases (note: although these pairs are not physically possible, the difference between ResISDR and MaxISDR is not large, distributed around the boundary line). In seismic risk assessment, such unrealistic cases can be avoided by simply adopting physically possible MaxISDR-ResISDR pairs only.

3.5. Seismic loss ratios based on IM-DV functions

To verify that the fitted statistical models produce reasonable estimates of the EDP parameters, the developed multi-variate seismic demand models (i.e. IM-EDP functions) as well as story-based damage-loss models (i.e. EDP-DV functions) are implemented in Monte Carlo simulation. The adopted EDP-DV functions are based on the study by Ramirez and Miranda (2009). These EDP-DV functions involve three loss generation modes, i.e. collapse, demolition, and non-collapse damage, which are determined based on MaxISDR, ResISDR, and PFA at different story levels. The collapse is assessed using the collapse fragility model that is developed as part of IDA (i.e. relationship between collapse probability and seismic excitation level; the results are omitted). On the other hand, the demolition failure mode is determined by comparing ResISDR with the uncertain limit state (capacity) function that is approximated by the lognormal distribution (Ramirez and Miranda 2009). The parameters of the limit state function for demolition are the median and dispersion, expressed in terms of ResISDR. Typical ranges of the median and dispersion parameters are 0.02 to 0.05 and 0.3 to 0.6, respectively. The non-collapse damage is characterized by the IDA-based multi-variate seismic demand models (Section 3.4). Further descriptions can be found in Tesfamariam and Goda (2015).

The mean IM-DV functions are developed by generating 10,000 samples of normalized seismic damage costs for each IM level. Figure 5 compares two sets of IM-DV functions for collapse, demolition, non-collapse damage, and total loss with the demolition damage limit state parameters = [0.015, 0.3] and [0.03, 0.3]. The influence of the median demolition limit state parameter on the IM-DV functions for demolition is significant, as illustrated in the figures. When the median limit state for demolition is small, the demolition failure mode consists of about 30% of the entire failures (peaked at around IM equal to 0.22 *g*). When the limit state function is more uncertain (dispersion is increased from 0.3 to 0.5; results are omitted), the demolition loss increases by about 10-40%, depending on the IM level.

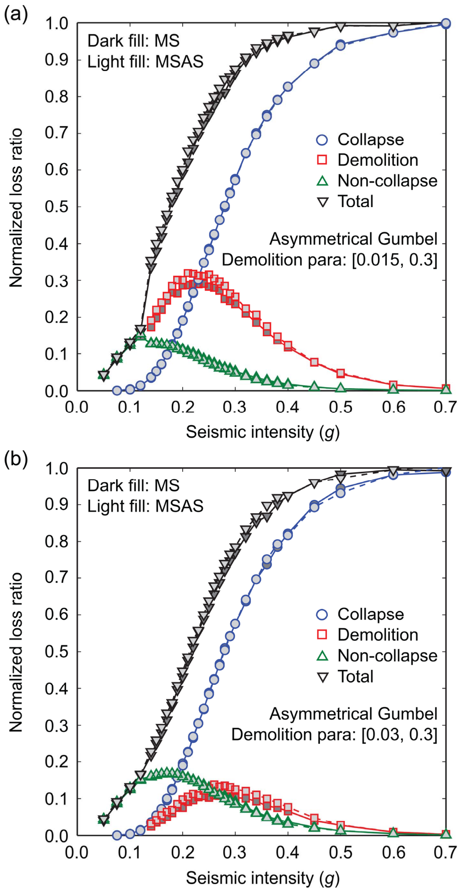

Figure 5: IM-DV functions for collapse, demolition, non-collapse damage, and total loss by considering two sets of demolition damage state limit parameters: (a) [0.015, 0.3] and (b) [0.03, 0.3].

3.6. Seismic loss estimation

Finally, to investigate the effects of aftershocks and individual contributions due to different failure modes on the estimated seismic loss, quantitative seismic loss estimation is carried out using the developed multi-variate seismic demand models for the 4-story non-ductile RC building. Seismic loss estimation of the 4-story non-ductile RC building is carried out using Monte Carlo simulation by generating seismic loss samples for numerous synthetic seismic events, obtained from regional PSHA in Victoria. Detailed descriptions of the seismic loss estimation can be found in Tesfamariam and Goda (2015).

Four seismic loss curves (i.e. plot of seismic loss as a function of annual probability) are obtained by considering MS-based and MSAS-based seismic demand models with and without demolition, and are shown in Figure 6. The aftershocks increase the seismic loss slightly (about 1 to 4%), indicating that the overall aftershock effects are relatively minor. The consideration of the demolition failure modes increases the seismic loss curve noticeably (about 4 to 8%). The breakdown of the total seismic loss for different failure modes depends on the probability level; at return period levels shorter than 1000 years, dominant failure modes are non-collapse damage, while at longer return period levels, collapse and demolition failures become more frequent. For MSAS-based demand models, at the 1000-year return period level, 4.8%, 0.8%, and 94.4% of the failures are due to collapse, demolition, and non-collapse damage, respectively. At the 10,000-year return period level, these percentages change to 83.4%, 16.6%, and 0.0% for the collapse, demolition, and non-collapse modes, respectively.

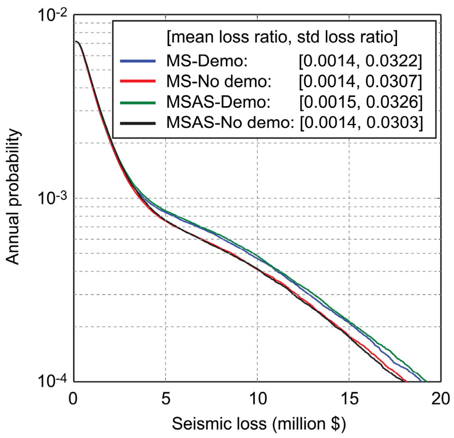

Figure 6: Seismic loss curves for MS records and MSAS sequences with and without demolition failure cases. The values shown in the parentheses are the mean and standard deviation (std) of the loss ratios in terms of total replacement cost (= $12.8 million).

4. CONCLUSIONS

This study developed IDA-based multi-variate seismic demand models using copulas, and applied them to seismic risk assessment of a 4-story non-ductile RC frame in Victoria, BC, Canada. It considered not only mainshock hazards but also threat posed by major aftershocks. To reflect regional seismicity in Victoria, up-to-date regional seismic hazard information as well as ground motion data was taken into account as part of model development (e.g. multiple-CMS-based record selection and updated ground motion database for MSAS sequences that includes records from the 2011 Tohoku earthquake). The seismic risk assessment took into account multiple damage-loss generation modes due to non-collapse damage, collapse, and demolition, which were evaluated in terms of inter-related EDP parameters, i.e. MaxISDR, ResISDR, and PFA. The copula method captures upper tail and nonlinear dependence of key seismic demand variables and facilitates the separate modeling for marginal probability distributions and dependence functions. Therefore, it is suitable for characterizing EDP variables with heavy right tail whose marginal distributions cannot be represented by the normal or lognormal distribution (e.g. ResISDR). From the methodological viewpoints, the proposed method is an extension of current IDA-based uni-variate seismic demand models to multi-variate models.

The main results from the current investigations are:

- Joint probabilistic modeling of MaxISDR, ResISDR, and PFA was implemented successfully by adopting the Frechet distribution for MaxISDR and PFA and the generalized Pareto distribution for ResISDR, while by adopting the asymmetrical Gumbel copula function for MaxISDR-ResISDR data pairs (note: PFA was modeled independently for the structural model).

- The IDA results indicated that aftershock effects were more noticeable for ResISDR than MaxISDR. This led to moderate influence of major aftershocks on seismic loss caused by demolition. However, when the seismic demand models were integrated with seismic hazard, the aftershock effects were relatively minor in terms of overall seismic loss (1 to 4% increase).
- The effects of multi-variate seismic demand modeling on seismic loss were significant. The critical information is the demolition limit state curve.

5. ACKNOWLEDGEMENTS

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