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FRAGILITY CURVES FOR SEISMIC VULNERABILITY ASSESSMENT OF CONCRETE SHEAR WALL BUILDINGS IN WESTERN CANADA

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ABSTRACT: Structural damage observed during past earthquakes reveals that a large number of older buildings designed prior to the enactment of modern seismic codes are vulnerable to strong earthquakes. Depending on the year of construction, the level of seismic design and detailing, as well as the effects of structural irregularities, a large inventory of building infrastructure in Canada remains seismically vulnerable. Vulnerability assessment of these buildings often requires a probabilistic approach due to the probabilistic nature of seismic hazards. Fragility curves have been developed in the current research project for seismic risk assessment of concrete shear wall buildings in western Canada. The first phase of the project, presented in this paper, involves shear wall buildings conforming to the requirements of the current National Building Code of Canada (NBCC 2010) with regular floor plans. Subsequent phases will include the effects of irregularities on seismic vulnerabilities, as well as the year of construction. In addition, concrete frame and masonry buildings in eastern and western Canada will be included. A reference shear wall building was first designed for the development of fragility curves based on the seismic provisions of NBCC 2010 for Vancouver. A set of twenty earthquake records, compatible with the design response spectra, was selected to conduct 3D nonlinear dynamic time history analyses using Perform 3D software. Spectral acceleration was selected as the seismic intensity parameter. The inter-storey drift of the first floor was selected as an engineering demand parameter. Incremental Dynamic Analysis (IDA) was employed with different scale factors to capture the structural behaviour under different levels of seismic excitations. Fragility curves were generated as a visualized probabilistic tool to assess seismic performance of shear wall structures using the IDA results. The curves depict probability of exceedance for different limit states, consisting of immediate occupancy, life safety and collapse prevention.

1. Introduction

The objective of the current research project, which forms part of an overall research program on seismic risk assessment of reinforced concrete and masonry buildings at the University of Ottawa, is to develop fragility curves for shear wall buildings in western Canada. The paper presents fragility curves for representative midrise ductile shear wall buildings in Vancouver, conforming to the requirements of the National Building Code of Canada (NRCC 2010). Subsequent phases include seismic risk assessment of concrete shear wall buildings for different years of construction with irregularities in eastern and western Canada.

There has been limited previous research involving seismic risk assessment of shear wall buildings in Canada. Koduru and Haukaas (2009) modeled a 15 storey building with shear walls and gravity load frames. The researchers used fiber-discretized cross sections for plastic hinge regions of shear walls between the first story and the fourth story, and employed OpenSees software in 3D. They applied a probabilistic model for seismic hazard in Vancouver to perform loss assessment of a Vancouver high-rise building. Another research project, with emphasis on design of shear wall buildings was conducted by Ghodsi and Flores Ruiz. (2010) using the same software as that employed in the current investigation, i.e., Perform 3D. The researchers designed a 42-storey shear wall building located in Los Angeles, California. The first design of the building was based on a modal response spectrum analysis. The second design followed the Los Angeles Tall Building Design Council's prescribed methodology. The building was shown to satisfy service level design criteria using modal response spectrum analyses, and collapse prevention criteria using 7 time-history non-linear analyses using a fiber model for the shear wall.

The current research project involves the selection and design of a typical 5-storey concrete shear wall building for Vancouver and non-linear dynamic response history analysis of the building to generate fragility curves. The methodology followed and the results obtained are presented in the following sections.

2. Methodology Used for the Development of Fragility Curves

2.1 Selection of a Reference Building

A five-storey shear wall building, with a regular floor plan, was selected and designed based on the National Building Code of Canada (NBCC 2010), following the requirements for a fully ductile shear wall building in Vancouver. The seismicity of Vancouver was assumed representative of major urban centres in Western Canada. The building has 5 bays and two shear walls in each direction. An elastic 3D model of the building was first built to perform gravity and lateral load analysis under design loads. Once the analysis and design were completed, a 3D numerical model was constructed using the design quantities obtained to conduct dynamic inelastic response history analyses with computer software PERFORM 3D (2013).

PERFORM 3D is software developed by Computers and Structures, Inc. (CSI) for nonlinear analysis and performance assessment of 3D structures. It has variety of options for nonlinear modelling of elements in three dimensions, including plastic hinge models for beams and columns and fiber models for shear walls, which were employed to construct the building model. Numerical model of the five storey structure for PERFORM 3D is shown in Fig.1.

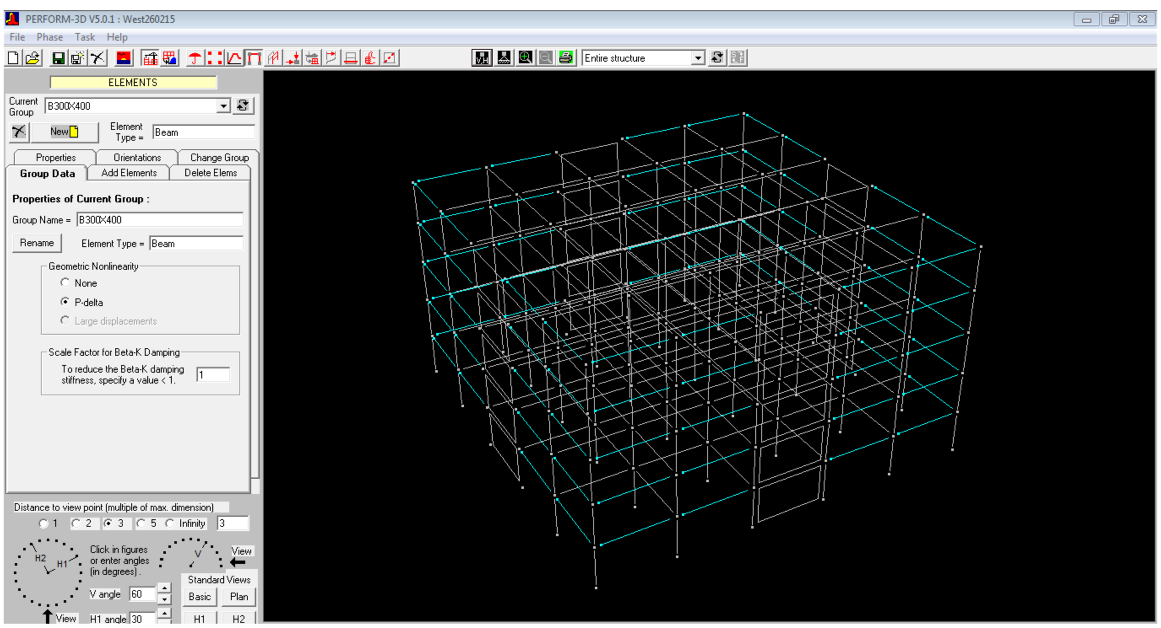

Fig. 1 – Numerical simulation of the five-storey building in PERFORM 3D

Nonlinear beam and column elements were modeled having lumped plasticity defined by plastic hinges. Nonlinear flexural characteristics of each member were defined by moment-rotation relationships. SAP2000 (2009) software was used to compute sectional analysis of elements to establish the properties of plastic hinges, which were then assigned at member ends. Strength decay characteristics of the primary moment-rotation relationship were specified based on the ASCE 41 (2007) recommendations, defining the onset of flexural failure in the member. The initial effective elastic member stiffness values were assumed to be 0.5EI and 0.70EI for beams and columns, respectively.

Because response of shear wall buildings is primarily governed by the behavior of the walls; it was essential to apply an accurate and detailed model for the shear wall. Therefore, a fiber-discretized model was employed to simulate the wall behaviour. Confined concrete was assumed in the wall boundary elements and unconfined concrete was used elsewhere in the wall, following the confined concrete model developed by Saatcioglu and Razvi (1992) and unconfined concrete model of Hognestad (1951). The confined concrete properties were computed for fully ductile sections with close spacing of transverse steel in boundary elements. In addition, nonlinear shear behaviour was considered and assigned to the walls based on the experimental results obtained at the Portland Cement Association (Oesterlee et. al 1976) for rectangular walls. This was felt necessary to provide realistic input for finding displacements in the high range of deformations.

2.2 Incremental Dynamic Analysis

Incremental dynamic analysis (Vamvatsikos and Cornell, 2000) is an accepted method for estimating structural performance under different levels of seismic loads. It involves subjecting a structural model to a set of ground motion records, each scaled to multiple levels of intensity, producing curves of response parameter versus intensity level. Incremental dynamic analysis (IDA) gives thorough understanding of the range of response versus the range of potential ground motion intensity. It provides structural behaviour under severe ground motion levels, beside the estimation of the dynamic capacity of global structural system.

2.1.1. Record Selection

An important step in the incremental dynamic analysis is the selection of ground motion records that are representative of the building site. In the current study, synthetic earthquake ground motions, generated by Atkinson (2009), were used. These records are compatible with the uniform hazard spectra (UHS) specified for seismic design in the 2005 National building code of Canada (NRC 2005), which are also applicable to 2010 NBCC hazard values. The records are for earthquakes having 2% probability of exceedance in 50 years. The "target" UHS depends on the location and the site condition, where the site condition is classified based on the time-averaged shear-wave velocities in the top 30 m of soil deposit. Atkinson applied the stochastic finite-fault method to generate earthquake time histories that may be used to match the 2005 NBCC UHS for a range of Canadian sites. The records are provided for soil types A, C, D, and E specified in the code (NRC 2010). In this study, the records generated for the reference soil type C were used.

The records are provided in four sets of 45 time histories: M6.5 at 10 to 15 km, M6.5 at 20 to 30 km, M7.5 at 15 to 25 km, and M7.5 at 50 to 100 km. From each of these four groups, 5 records were selected (twenty records in total) which matched the target spectrum in the period range of 0.4 to 2.5. These records had the lowest standard deviation for the ratio of simulated response spectra to the target UHS $(S_a)_{target}/(S_a)_{simulated}$ in the range of periods of interests. Fig.2 shows the comparison between the spectral accelerations of a sample record and the UHS for Vancouver.

2.1.2. Scaling of the Records

After selecting the ground motion records, the next step is to scale them to cover the entire range of structural response, from elastic behaviour, to yielding, and from post-yield behaviour to global dynamic instability. Analyses were performed at increasing levels of intensity measure (IM) until numerical non-convergence was observed, indicating global dynamic instability. Additional analyses were performed at intermediate IM-levels to bracket the range of response from inelasticity to global collapse (Vamvatsikos and Cornell, 2000).

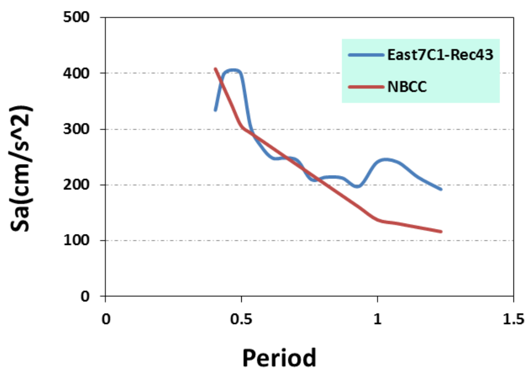

Fig. 2 – Comparison of spectral acceleration of a single record and UHS

2.1.3. Seismic Intensity Measure

Spectral accelerations for 5% of critical damping (S_a,5%) was selected as the measure of ground motion intensity. This measure of intensity reflects the characteristic of the earthquake, while also dependant on structural period, as opposed to sometimes used Peak Ground Acceleration (PGA), which only reflects a characteristic of an earthquake record. Moreover, S_a is a parameter defined in the National Building Code of Canada as a design parameter, frequently used by designers. However, S_a is sensitive to changes in period. It can have sharp local maximums and minimums. Therefore, instead of using one value for S_a, the average of two values were used as the ground motion intensity parameters. These were the spectral acceleration at the period computed on the basis of code recommended empirical equation and the spectral acceleration at 1.4 times the computed code period, which corresponded to the period of the current structure computed through analysis.

2.1.4. Engineering Demand Parameter

Incremental dynamic analysis involves the selection of a seismic intensity measure and an engineering demand parameter (EDP) that is representative of the response of the structure. The first storey horizontal drift ratio was selected in the current investigation as EDP. This implies that the inter-storey drift for the first storey was selected as a damage indicator. The use of inter-storey drift to define different limit states is quite common among the engineers, which can be computed and rationalized easily. It should be noted, however, that in a multi-storey shear wall building, where the primary mode of deformation is flexure, inter-storey drift in upper floors may show higher values even though the wall remains elastic, and hence may not reflect the anticipated state of damage..

3. Analysis Results

3.1 Result of IDA

The analytical model shown in Fig. 1, with the inelastic member properties discussed above, was used to conduct inelastic dynamic response history analysis under selected and scaled ground motion records. The use of a set of twenty records with thirteen different scale factors resulted in 260 dynamic analyses. The results are depicted in the form of incremental dynamic analysis (IDA) curves in Fig 3. The curves show the variation of first storey drift with incrementally increasing earthquake intensity in the form of spectral values.

3.2 Definition of Damage Limit States

IDA curves give a view of structural response under increasing earthquake intensity. They can be used to identify the damage states imposed on the building. The limiting states of damage first need to be defined before a judgement can be made on the damage experienced by the building. Three different damage

states were used, as defined by ASCE 41. These consist of i) Immediate Occupancy, ii) Life Safety and iii) Collapse Prevention. The description of these damage states, based on ASCE 41, is as follows:
Immediate occupancy: Building remains safe to be re-occupied; lateral-force and gravity-load-resisting systems retain most of their entire design strength;

Life Safety: Significant damage to the structure occurs; structural elements and components may be severely damaged; gravity-load-carrying elements continue functioning.

Collapse Prevention: Substantial damage to structural elements occurs; significant strength and stiffness degradation of the lateral-load-resisting system is imposed; large permanent lateral deformations of the structure are evident and the structure is not repairable; the structure is not safe to reoccupy.

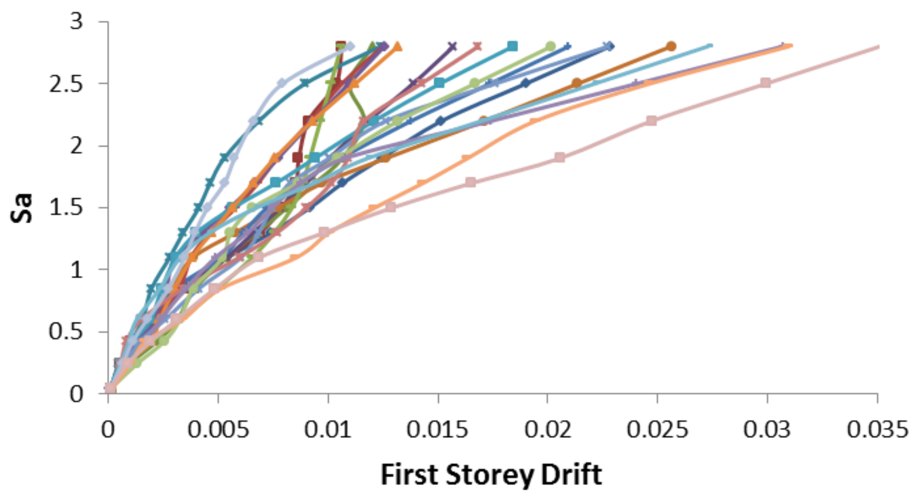

Fig. 3 – Result of Incremental Dynamic Analysis

The above definitions provide qualitative descriptions of damage states while quantitative values are required for the selected engineering demand parameter, i.e., first-storey drift. Following the ASCE 41 recommendation, 0.5% drift ratio was selected as the threshold for “immediate occupancy” for the shear wall building selected. Observations made from the IDA curves confirm that until this drift value is exceeded the structural behaviour remains essentially linear, allowing the building to be immediately re-occupied. For the collapse prevention limit state, ASCE 41 suggests a value of 2% drift for shear wall buildings. However, the collapse state indicated by the response history analysis made it necessary to re-assess this limit, because a wide range of drift ratios were observed at collapse under different earthquake records. Hence, the collapse was assumed to occur either, when the slope of the IDA curve becomes significantly reduced relative to the slope in the elastic range, or the numerical instability occurs in the analysis. More specifically, the change in drift value in the IDA curve within a step divided by the change in S_a over the same step increases beyond five times the same ratio for the linear step, or numerical instability is encountered in analysis, then the structure is assumed to have reached the collapse prevention state of damage. This definition was adopted from Jeong and Elnashai (2007) and can be rationalized as it indicates a significant change in drift with a small change in earthquake intensity. The numerical instability in analysis is attained when any one of the final conditions defined as failure in input data is reached or too many nonlinear events take place within the structure.

Using the above definition for collapse, the drift value of 1.7% was found to be appropriate for use as the collapse limit. The IDA values showed that 25% of the records resulted in collapse at a drift less than 1.7% drift, 50% reached collapse at a higher value, and 25% did not reach collapse until the scaling factor went up to S_a of 2.8, which was the maximum scaling used in analyses. Table 1 shows the analysis results at collapse for the applied twenty records compatible with western Canadian seismicity. Fig. 4 shows the relationship between the incremental dynamic analysis results and defined limit states.

Finally, half of the value for collapse prevention limit was assumed to be used as the threshold for life safety limit state (0. 85 % drift).

Table 1 – Observation of collapse condition for different records

		Drift Observed at Collapse Condition	Related Spectral Acceleration(g)
M6.5 at 10-15 km	W1	0.019	2.5
	W2	Before Observation of Collapse	-
	W3	Before Observation of Collapse	-
	W4	0.016	2.8
	W5	0.012	2.8
M6.5 at 20-30 km	W6	>0.025	-
	W7	0.021	2.8
	W8	Before Observation of Collapse	-
	W9	0.01	1.9
	W10	Before Observation of Collapse	-
M 7.5 at 15-25 km	W11	0.0185	
	W12	Before Observation of Collapse	-
	W13	0.023	2.8
	W14	0.014	2.5
	W15	0.02	2.8
M 7.5 at 50-100 km	W16	0.024	2.5
	W17	0.022	2.5
	W18	0.025	2.5
	W19	0.011	2.8
	W20	0.021	1.9

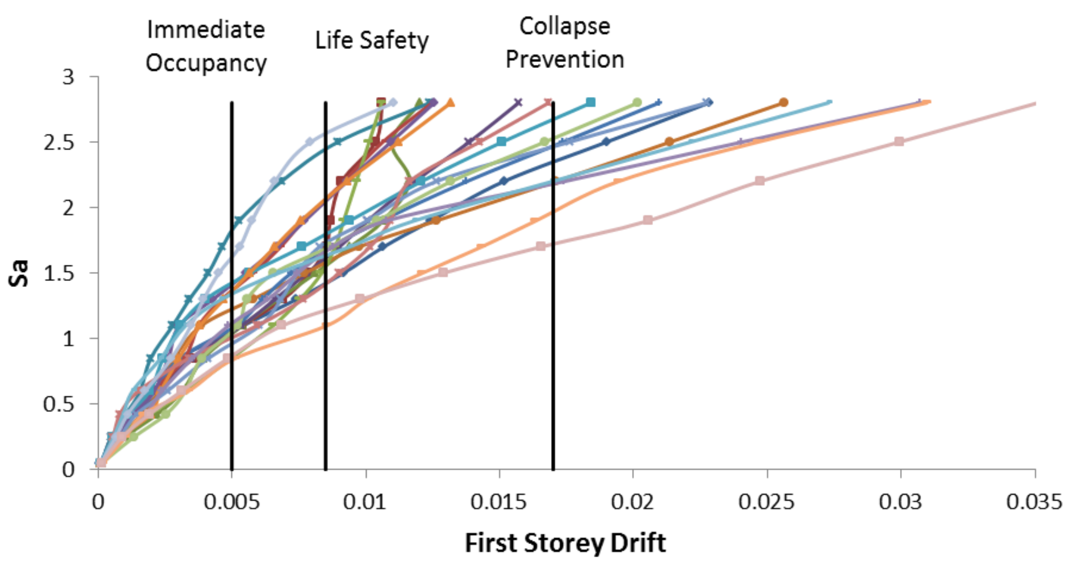

Fig. 4 – IDA results and limit states

4. Fragility Analysis

4.1 Role of Fragility Analysis in Seismic Risk Assessment

IDA is a powerful analysis method, which provides input in a probabilistic framework to estimate the annual likelihood of the event that the demand exceeds the limit-state (Immediate Occupancy, Life Safety and Collapse Prevention). Pacific Earthquake Engineering Center (2000) gives a summarized equation for the probabilistic seismic risk assessment framework. This framework suggests a relationship between a certain key decision variable, DV, such as annual seismic loss and a series of conditional probabilities, deaggregating mean annual frequency (MAF) of exceeding the decision variable, λ_{DV}, in terms of damage measure, DM, and ground motion intensity measure, IM given in Eq. 1.

$$\lambda_{DV} = \iint G(DV \mid DM)dG(DM \mid IM)d\lambda_{IM} \tag{1}$$

Where, G(DV/DM) is the probability that the decision variable exceeds specified values given that the engineering damage measures (first storey drifts) are equal to particular values, G(DM/IM) is the probability that the damage measures exceed these values given that the intensity measure (spectral acceleration at the fundamental period) is equal particular values, and λ_{IM} is the MAF of the ground motion intensity measure. The part G(DV/DM) in the deaggregated equation is called seismic fragility function. Seismic fragility relates the probability of reaching or exceeding predefined levels of damage to the severity of ground motion intensity. Analytical fragility curves are derived from the results of numerical simulations of the structure under artificial or historical earthquake records. Fragility function is described in Eq. 2.

$$P[D > Di \mid IM] = \phi\left[\frac{\ln(x / Di)}{\sqrt{\beta_{d/IM}{}^{2} + \beta_{c}^{2} + \beta_{m}{}^{2}}}\right] \tag{2}$$

In which Φ(.) is the standard normal cumulative distribution function, Di is the upper bound for each damage level, x is the median value of demand as the function of IM, $\beta_{d/IM}$ is the dispersion (logarithmic standard deviation) of the demand conditioned on the IM, β_c is the capacity uncertainty and β_m is the modeling uncertainty. Herein, the value of 0.5 is assumed for the part Sqrt($\beta_c^2 + \beta_m^2$) and β_{IM} is calculated based on dispersion of results of twenty records used in IDA for each scale factor.

4.2 Derived Fragility Curves

Based on the Performance Based Earthquake Engineering (PBEEE) methodology of the Pacific Earthquake Engineering Center, described above in Section 4.1, using the results of IDA, probability of exceeding limit states of immediate occupancy, life safety and collapse prevention (defined in Section 3.2) were calculated for different levels of spectral accelerations ($S_a(T_1)$). Fig. 5 shows the fragility curves of these three limit states for the five-storey shear wall building designed conforming to NBCC 2010 requirements for fully ductile buildings in Vancouver under western Canadian seismicity.

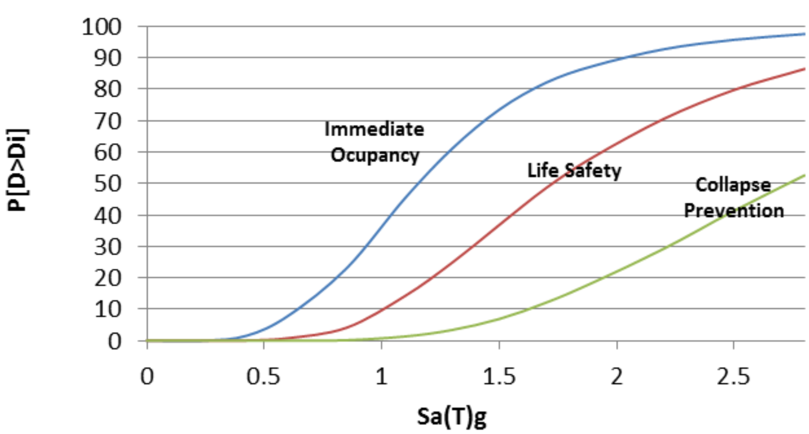

Fig. 5 – Fragility curves derived for a fully ductile 5-storey shear wall building in Vancouver

5. Summary and Conclusions

Due to the uncertainty and randomness inherent in seismic events, a probabilistic approach is often taken into account for assessing seismic behaviour of structures. Fragility analysis is used as a powerful tool for quantifying the likelihood of structural damage as a function of ground motion intensity. A five-storey shear wall building was designed as a fully ductile structure for Vancouver, BC based on 2010 NBCC, and assessed through fragility analysis. Twenty synthetic records generated for Western Canadian seismicity were selected and applied with thirteen different scale factors to cover the entire range of structural behaviour using Incremental Dynamic Analysis. Spectral Acceleration at the fundamental period of the structure was selected as the ground motion intensity measure, and the first storey drift was assumed to be the engineering demand parameter, representing the level of damage to the structure.

A challenging task was to define limit states consistent with the observed behaviour of the numerical model, resulting in values of 0.5%, 0.85% and 1.7% for Immediate Occupancy, Life Safety and Collapse Prevention, respectively, based on the result of IDA. Probabilistic analysis of the results of IDA is plotted in the form of fragility curves for the three limit states defined above. For a certain value of spectral acceleration, one can establish the probability of passing different limit states for buildings having similar design parameters as those for which the curves were generated.

The uniform hazard spectrum (UHS) in the National Building Code (NBCC 2010) results in a spectral acceleration of 0.65g when the empirical code expression is used for the fundamental period, and 0.55g when the period was computed using dynamic analysis of the representative five-storey shear wall building in Vancouver. The results indicate that NBCC-2010 objective of providing life safety under UHS hazard level is met, as the probability of exceeding life safety limit at Spectral Accelerations between 0.55g and 0.65g for the building is close to zero. Moreover, regular mid-rise shear wall buildings designed for Vancouver based on NBCC-2010 are not prone to reach the collapse state, under probable range of seismic hazard levels for this city.

The methodology described in the paper is useful for evaluating seismic behaviour of existing buildings. The current phase of the research project is intended to assess the behaviour of fully ductile regular mid-rise shear wall buildings in Western Canada. In the next phase, the research is aimed at extending the development of fragility curves for different shear wall buildings depending on the year of construction, the level of seismic design and detailing, and structural irregularities, both for eastern and western Canada, in order to have a comprehensive set of assessment tolls for all shear wall buildings in Canada.

6. References

ASCE. “Seismic rehabilitation of existing buildings.” ASCE 41, ASCE, Reston, VA, 2007..

ATKINSON, Gail M. "Earthquake time histories compatible with the 2005 National building code of Canada uniform hazard spectrum." *Canadian Journal of Civil Engineering*, Vol. 36, No. 6, 2009: 991-1000.

CORNELL, C. Allin., KRAWINKLER, Helmut. “Progress and challenges in seismic performance assessment,” *PEER Center News* **3** (2) http://peer.berkeley.edu/news/2000spring/index.html, 2000.

CSI (Computers & Structures, Inc.). “SAP2000 Version 14. Structural Analysis Program.” CSI: Berkeley, CA., 2009.

CSI (Computers & Structures, Inc.). “Perform 3D Version 5.0.1.” Nonlinear Analysis and Performance Assessment of 3D Structures. CSI: Berkeley, CA.,2013.

GHODSI, Tony, FLORES RUIZ, Jose A. "Pacific earthquake engineering research/seismic safety commission tall building design case study 2." *The Structural Design of Tall and Special Buildings*, Vol. 19, No. 1-2, 2010: 197-256.

HOGNESTAD, E. ''A study of combined bending and axial load in reinforced concrete members.'' *Bull. Ser. No. 399*, University of Illinois, Engineering Experimental Station, Urbana, Ill., 128,, 1951.

JEONG Seong-Hoon, ELNASHAI Amr S. "Probabilistic fragility analysis parameterized by fundamental response quantities". *Engineering Structures*, Vol. 29, No. 6, 2007, pp. 1238–51.

KODRU, Smitha Devi, HAUKASS Terje. "Probabilistic seismic loss assessment of a Vancouver high-rise building", *Journal of structural engineering,* Vol. 136, No. 3, 2009, pp. 235-245.

National Research Council of Canada (NRCC). "National Building Code of Canada," Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON, 2005.

National Research Council of Canada (NRCC). "National Building Code of Canada," Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON, 2010.

OESTERLE, R. G. et al. "Earthquake resistant structural walls*:* Tests of isolated walls." Construction Technology Laboratories, *Portland Cement Association*, 1976.

SAATCIOGLU, Murat, RAZVI, Salim R.. "Strength and ductility of confined concrete." *Journal of Structural Engineering*, Vol. 118, No. 6, 1992: 1590-1607.

VAMVATSIKOS, Dimitrios, CORNELL, C. Allin. "Incremental dynamic analysis." *Earthquake Engineering & Structural Dynamics*, Vol 31, No. 3, 2002: 491-514.

ZAREIAN, Farzin, KRAWINKLER Helmut, "Structural system parameter selection based on collapse potential of buildings in earthquakes." *Journal of structural engineering* 136, No. 8, 2010: 933-943.