17th World Conference on Earthquake Engineering, 17WCEE

Sendai, Japan - September 27th to October 2nd, 2021

SEISMIC FRAGILITY ANALYSIS OF BUILDINGS IN CANADA

M. Saatcioglu[(1)], Y. R. Nazari[(2)], A. Al-Mamun

[(1)] *Distinguished University Professor, University of Ottawa, Murat.Saatcioglu@uOttawa.ca*
[(2)] *Former PhD Student, University of Ottawa, yasemin.rafi@gmail.com*
[(3)] *Former PhD Student, University of Ottawa, amamu022@uottawa.ca*

Abstract

Seismic design requirements in the National Building Code of Canada (NBC) have evolved and improved since its inception in 1941. While buildings designed using recent editions of the NBC are expected to perform well during seismic events, older buildings designed prior to the enactment of modern seismic codes remain vulnerable to earthquakes. Seismic fragility curves were developed for representative reinforced concrete frame and shear wall buildings in Canada for vulnerability assessment, relating seismic hazard to probability of exceeding pre-selected performance levels. The curves were generated for moderately ductile buildings in eastern Canada and fully ductile buildings in western Canada. Six different reinforced concrete buildings with three different heights (2-storey, 5-storey and 10-storey) and two seismic force resisting systems (frame and shear wall) were considered in each region. Both structural systems consisted of regular floor plans having 5 bays in each direction with a span length of 7.0 m. Incremental dynamic analysis (IDA) was conducted using computer software PERFORM-3D and 20 earthquake records compatible with the 2010 NBC Uniform Hazard Spectra for Ottawa and Vancouver, with different scale factors resulting in over 200 dynamic inelastic analyses for each building model. Inter-storey drift ratio was adopted as the damage indicator. Three damage states were considered corresponding to three performance levels as commonly accepted performance limits; i) Immediate Occupancy, ii) Life Safety, and iii) Collapse prevention. The Immediate Occupancy performance level describes the damage state where structure is safe to be re-occupied having suffered minor damage to the structural elements with minor spalling and flexural cracking. The inter-storey drifts of 1% and 0.5% were considered for this limit state for frame and shear wall buildings, respectively. The Life Safety performance level describes the damage state where significant damage has occurred to the structure with extensive cracking and hinge formation in primary structural elements, while maintaining life safety of the occupants. The inter-storey drifts of 2% and 1% were adopted for frame and shear wall building for this limit state. The Collapse Prevention performance level describes the damage state where structure is at the onset of partial or total collapse with extensive cracking, hinge formation and reinforcement buckling in structural elements. The median value of maximum inter-storey drift demands for all records was considered as CP performance level. The results are presented in the paper.

Keywords: fragility analysis; performance levels; reinforced concrete; seismic assessment; seismic vulnerability.

1. Introduction

Seismic hazard in Canada can be characterized by the seismicity of two distinct regions; eastern Canada and western Canada; with a relatively stable continental shelf between the two. Significant seismic activities occur in western Canada because of the presences of active faults along the Pacific Rim. Geological Survey of Canada records more than 1000 earthquakes in western Canada with more than 100 earthquakes of magnitude 5 or greater. Seismic activity in eastern Canada occurs with reduced frequency of approximately 500 earthquakes per year, with on the average three magnitude 5 earthquakes taking place per decade [1]. Eastern Canada does not have active faults. The earthquakes in this region are believed to be related to the regional stress fields with earthquakes concentrated in regions of crustal weakness. Stronger earthquakes are expected in the west, though damaging earthquakes have also occurred in the east. This difference in seismic regions is reflected in building design practices that follow the requirements of the National Building Code of Canada.

The building inventory in Canada can be viewed in two broad groups; those designed prior to the enactment of modern seismic codes, and those designed using the more recent seismic hazard values and building design and detailing practices. The design base shear equation in NBCC has changed since the inception of seismic provisions in 1941 [2]. Earlier equations defined seismic base shear as a percentage of seismic weight of building as seismic coefficient. In the 1953 NBC [3], the building height was introduced as a design parameter, crudely reflecting the effect of building period on seismic coefficient. The hazard values were introduced in 1953 through seismic maps with seismic zones for different regions. In the 1965 NBC [4], the differences in construction type and associated level of ductility were introduced through coefficient C, reducing base shear for reinforced concrete frame and shear wall buildings with detailing for ductile response, while increasing the base shear for other non-ductile buildings. In the 1970 NBC [5] the hazard values were revised. The effect of construction type was treated more extensively through coefficient K, reflecting the associated level of ductility. Empirical expressions were also introduced for the computation of fundamental period. This was followed by the 1975 NBC [6] Commentary with ductility factors for different building types for use in dynamic analysis. The requirements remained essentially the same in the 1980 NBC [7] with refinements made to seismic response coefficient S as affected by fundamental period. New seismic zoning maps were introduced in the 1985 NBC [8] with seismic velocity and acceleration ratios specified for each zone, refining hazard values significantly based on 10% probability of exceedance in 50 years. Further refinements were introduced to the seismic response coefficient S with a new empirical period equation provided for shear wall buildings. The ductility related construction type factor K was replaced by force modification factor R in 1990 NBC [9], with a calibration factor U, which introduced a reduction in base shear to account for structural over-strength and to bring the force level to the same level of safety implied in earlier codes. The same base shear expression remained essentially the same until 2005 with a revised empirical equation introduced for fundamental period of shear wall buildings. Significant changes were introduced in 2005 [10] with new site-specific uniform hazard spectra having 2% in 50 year probability of exceedance. The approach was kept the same in the 2010 NBC [11] with new hazard values introduced in the 2015 NBC. The hazard values in the 2015 NBC [12] are 17% to 28% higher for Vancouver and 24% to 15% lower for Ottawa relative to those in the 2010 NBC [11] within the 0.5 sec to 1.0 sec building period range.

The design and detailing requirements for reinforced concrete buildings in CSA A23.3 went through a similar evolution. There were no seismic design requirements prior to CSA A23.3-1973 [13], which was referenced in the 1975 NBC [6]. Ductile design and detailing requirements for seismic resistance were introduced for the first time in 1973, which remained the same until 1984 [14]. Significant improvements were made to the standard in 1984 [14] with the introduction of capacity design requirements, protecting critical elements and preventing non-ductile failures. Three levels of seismic detailing were specified for the first time for: i) ductile response, ii) moderately ductile response, and iii) frame members that are not part of the seismic resisting system but “go for the ride” during seismic response. Critical elements in ductile buildings were protected and non-ductile failure modes were prevented by increasing design to levels that are associated with the development of probable moment resistances in plastic hinges at 125% of the steel yield strength. The same capacity design concept was implemented in nominally ductile buildings using nominal capacities. The stringency of design depended on the design ductility demand selected in the 1985 NBC [8], which made reference to CSA A23.3-1984 [14]. Hence, 1985 was taken as the "benchmark" year for significant improvements in seismic design of reinforced concrete buildings in Canada. The same year was adopted as the benchmark year in the Canadian Seismic Screening Manual [15].

It is economically not feasible to replace seismically deficient buildings with new and improved buildings. Therefore, the best seismic risk strategy is to conduct seismic vulnerability analysis and implement appropriate seismic retrofit techniques. It is preferable to conduct seismic vulnerability analysis of buildings through dynamic inelastic response history analysis. However, this may not be feasible for the majority of buildings. An alternative is to conduct fragility analysis using fragility curves that incorporate design characteristics of the building being assessed. Fragility analysis provides a probabilistic methodology for assessing seismic vulnerability of existing buildings. It can be conducted using fragility curves that provide probability of exceeding pre-determined performance levels as a function of earthquake intensity for a given region and for a building type with certain characteristics [16]. The fragility curves can serve as convenient tools for decision makers to take appropriate seismic risk mitigation strategies. The objective of this paper is to present seismic fragility curves for reinforced concrete frame and shear wall buildings in Canada, designed after the benchmark year of 1985. Twelve buildings with three building heights were selected for this purpose. The buildings had seismic force resisting systems consisting of either reinforced concrete frames or shear walls, each system having 2-storey, 5-storey and 10-storey building heights. The buildings were either designed for Ottawa, representing buildings in a moderate seismic region of eastern Canada, or for Vancouver, representing buildings in a strong seismic region of western Canada.

2. Selection of Buildings

Three frame and three shear wall buildings, without having any irregularities, with 2-storey, 5-storey and 10-storey heights were selected for Ottawa, representing eastern Canadian seismicity, and for Vancouver, representing western Canadian seismicity. Figures 1 and 2 show the plan and elevation views of the buildings. All the buildings were designed using normal density concrete (w_c = 24000 kg/m^3). The frame buildings were designed for office occupancy with live load of 2.4 kPa and the shear wall buildings were designed as residential buildings with 1.9 kPa live load. The superimposed deadloads, consisting of floor finish, partition walls and mechanical/electrical fixtures, were 1.33 kPa and 1.0 kPa for the frame and shear wall buildings, respectively. The concrete used was in the normal-strength range with concrete strength varying between 30 MPa and 40 MPa (f'_c = 30 MPa for the 5-storey frame buildings, 35 MPa for all shear wall buildings and 40 MPa for the 2-storey and 10-storey frame buildings). The reinforcement yield strength was 400 MPa for all cases.

The buildings were designed based on the 2010 NBC [11] seismic requirements with the accompanying CSA Standard A23.3-04 "Design of Concrete Structures" [17] used for proportioning and detailing of members. The equivalent static load approach was used to compute elastic seismic base shear (V_e). The buildings were designed for normal occupancy with an importance factor of I =1.0, on firm soil (Soil Class C). The fundamental period was computed by performing Eigen Value analysis with reduced section properties according to CSA A23.3-04 [17]. The resulting fundamental periods were compared with those obtained empirically by the expressions given in the 2010 NBC [11], which requires the fundamental period to be used in equivalent static force method was not to be longer than those computed by the code-recommended empirical values by more than 150% in the case of frame buildings and 200% in the case of shear wall buildings. The building periods selected for use in design were selected with this provision of the NBC. Table 1 provides the fundamental period for each building and the corresponding design spectral accelerations for each building. The equivalent elastic static base shear V_{ed} is obtained as the product of the spectral acceleration at design period, soil modification factor, importance factor I_E, and the structural mass. The inelastic design base shear V_d is then obtained by dividing V_{ed} by ductility and over-strength related force modification factors, R_d and R_o. The force modification factors are also listed in Table 1. It should be noted that the building designs for Vancouver and Ottawa resulted in the same member sizes. This is because the ratio of seismic hazard values between Vancouver and Ottawa is approximately proportional to the ductility-based force modification factors for fully ductile buildings in Vancouver and moderately ductile buildings in Ottawa, thereby offsetting the effects of higher hazard values for Vancouver.

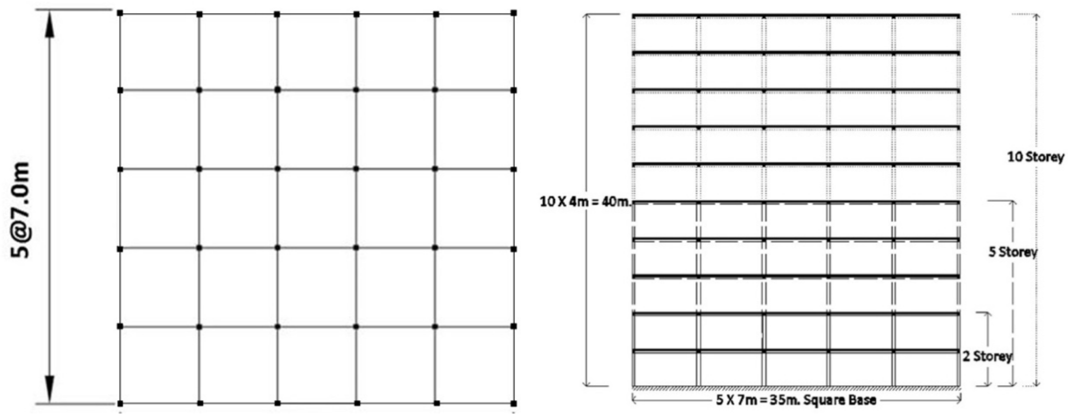


Fig. 1 – Plan and elevation views of frame buildings

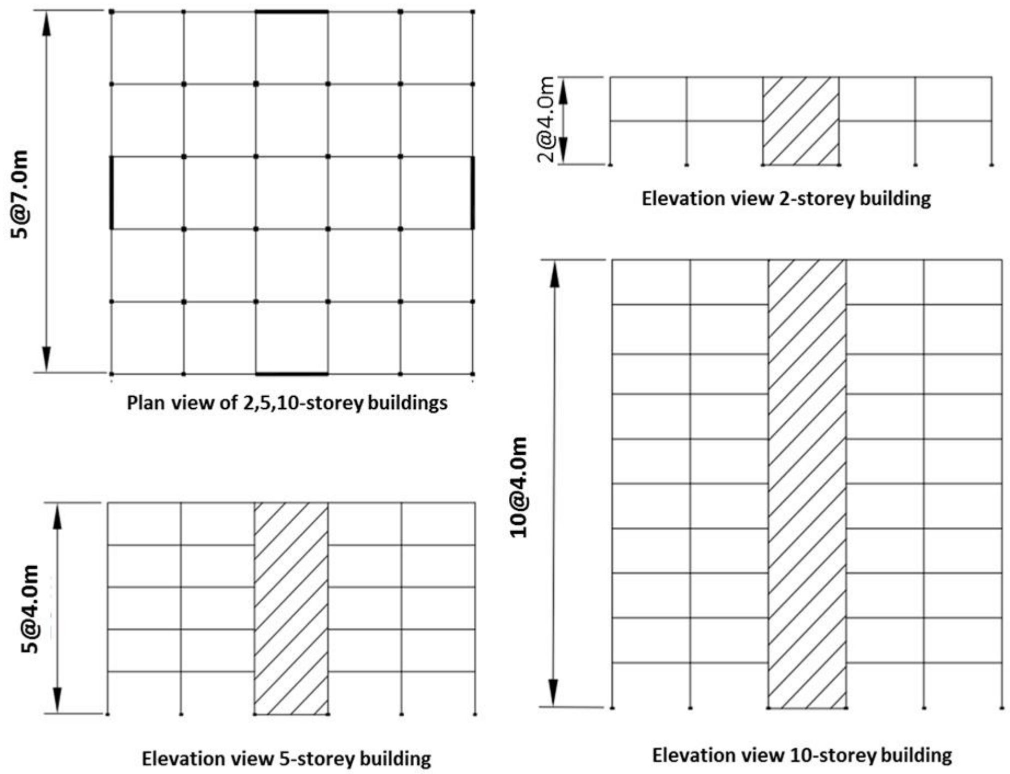


Fig. 2 – Plan and elevation views of shearwall buildings

3. Incremental Dynamic Analysis

The development of fragility curves for seismic vulnerability assessment involves a large number of response history analyses under incrementally changing earthquake intensity. This can best be handled through Incremental Dynamic Analysis (IDA) proposed by Vamvatsikos and Cornell [18]. Computer software PERFORM-3D [19] was selected to conduct nonlinear dynamic analysis to obtain the IDA results. PERFORM-3D is specialized software for damage assessment, specifically intended for performance-based seismic assessment of structures. The software permits monitoring of inelastic behavior of structural components with different levels of deformability.

IDA is a powerful method of estimating structural performance under incrementally changing intensities of earthquakes. The results are presented in the form of IDA curves, which show the variation in the selected response parameter with seismic intensity level. Choosing several records that are compatible with building site enables probabilistic analysis of IDA results, which can be presented in the format of seismic fragility curves. IDA gives thorough understanding of the range of structural response within the range of potential ground motion intensities. It also provides structural behaviour until failure, in addition to providing the estimation of dynamic capacity of the global structural system.

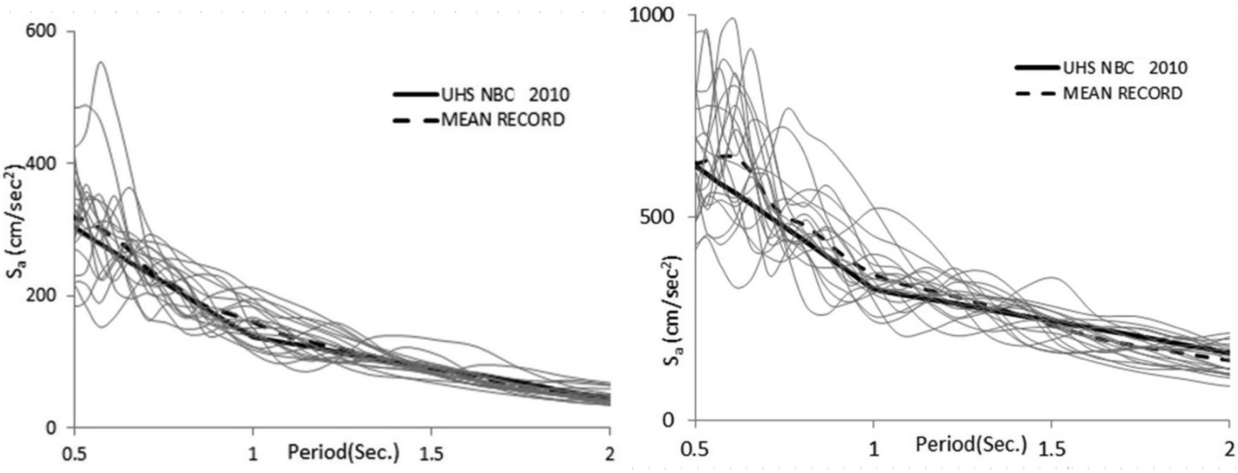
Table 1 – Design periods and spectral accelerations

Building	NBC period (sec)	Dynamic period (sec)	Design period (sec)	Ottawa			Vancouver		
				S_a	R_d	R_o	S_a	R_d	R_o
2-Storey frame	0.36	1.08	0.54	0.30g	2.5	1.4	0.62g	4.0	1.7
5-Storey frame	0.71	2.04	1.06	0.13g	2.5	1.4	0.32g	4.0	1.7
10-Storey frame	1.19	2.84	1.79	0.07g	2.5	1.4	0.20g	4.0	1.7
2-Storey shear wall	0.24	0.53	0.48	0.32g	2.0	1.4	0.65g	3.5	1.6
5-Stoery shear wall	0.47	0.82	0.82	0.21g	2.0	1.4	0.46g	3.5	1.6
10-Storey shear wall	0.80	1.72	1.59	0.09g	2.0	1.4	0.24g	3.5	1.6

2.1 Selection and scaling of earthquake records

In order to perform IDA, a set of ground motion records that are representative of the building site is needed. In the current project, artificial earthquake ground motions generated by Atkinson [20] were used. These records are compatible with the uniform hazard spectra (UHS) specified for seismic design in the 2005 NBC [10] and 2010 NBC [11] for earthquakes having 2% probability of exceedance in 50 years. The "target" UHS depends on the location and site conditions, where the site conditions are classified based on the time-averaged shear-wave velocity in the top 30 m of soil deposit (soil types A, C, D, and E specified in the building code). Atkinson applied the stochastic finite-fault method to generate earthquake time histories that may be used to match the 2005 NBC [10] UHS for a range of Canadian sites and different soil types. In this study, the records generated for reference soil conditions (Type C) were used.

The earthquake records were provided in four sets of 45 time histories for western Canada where each set corresponded to a different magnitude(M) distance combination: 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; and four sets of 45 time histories for eastern Canada: M6 at 10 to 15 km, M6 at 20 to 30 km, M7 at 15 to 25 km, and M7 at 50 to 100 km. Five records were selected from each of these eight sets, resulting in twenty records for each site, which were scaled to match the target spectrum in the period range of 0.5 to 2.5 for eastern and western Canada. These records were selected to have the lowest standard deviation for the ratio of simulated response spectra to the target UHS $(Sa)_{target/}(Sa)_{simulated}$ in the range of periods of interests. Fig. 3 shows the comparison of response spectra for the twenty records selected for Ottawa and Vancouver with the corresponding UHS given in NBC 2010 [11]. For IDA, the selected ground motion records needed to be scaled to cover the entire range of structural response, from elastic behaviour to yield, and from yield to structural failure.



(a) (b)

Fig. 3 – Comparisons of UHS and spectral accelerations of selected records for (a) Ottawa and (b) Vancouver

2.2 Seismic intensity measures and engineering demand parameters

Incremental dynamic analysis involves selecting two parameters: i) seismic intensity measure and ii) engineering demand parameter. The seismic intensity measure selected in the current paper is 5% damped spectral acceleration at design fundamental period $S_a(t_d)$. $S_a(t_d)$ was used by previous researches, including Vamvatsikos and Cornell [18] and Ellingwood *et al.* [21]. This measure of intensity reflects the characteristic of the earthquake, while also dependent on the structural period, as opposed to Peak Ground Acceleration (PGA), which is sometimes used. Moreover, S_a is a parameter defined in the National Building Code of Canada as a design parameter, and frequently used by designers.

The engineering demand parameter (EDP) is also needed for IDA to represent the damage state of the building. The first storey horizontal drift ratio was selected as EDP, which indicates the level of damage. The use of inter-storey drift to define different limit states is quite common among the engineers as it can be computed and rationalized easily. Furthermore, the first storey generally experiences higher level of plastic deformations under predominantly flexural response, with storey drift becoming a preferred EDP.

2.3 Damage limit states

The IDA curves quantify structural response in terms of first storey drift ratios. However, limit states for lateral drift need to be defined to assess the level of damage in a building. Three different limit states have been adopted to express the state of damage in buildings: i) Immediate Occupancy, ii) Life Safety and iii) Collapse Prevention. These limit states are the same as those described in ASCE 41 [22] as follows:

- *Immediate occupancy(IO):* Building remains safe to be reoccupied; lateral-force and gravity-load-resisting systems retain most of their design strengths.
- *Life safety(LS):* Significant damage has occurred to the structure; structural elements and components may be severely damaged; gravity-load-carrying elements continue functioning.
- *Collapse prevention(CP):* Substantial damage has occurred to structural elements; significant strength and stiffness degradation of the lateral-load-resisting system has taken place; large permanent lateral deformations were induced to the structure; the structure is not repairable and not safe to reoccupy.

The above provide qualitative descriptions of damage states while quantitative values are required to use the results of IDA. Following the ASCE 41 [22] recommendation, 0.5% and 1% storey drift ratios were selected as the threshold for immediate occupancy for shear wall and frame structures, respectively. Observations made from the IDA curves confirm that until this drift value is exceeded the structure remains essentially elastic, allowing the building to be immediately re-occupied. The life safety limit state was adopted from ASCE 41 [22] as 1% and 2% drift ratios of the first storey for shear wall and frame buildings, respectively. The analysis results indicate that the buildings remained intact at this drift level, while experiencing some limited inelasticity. This was found to be consistent with the intended damage level associated with life safety performance level. For the collapse prevention limit state, ASCE 41 [22] suggests a value of 2% and 4% drift ratios for shear wall and frame buildings, respectively. However, the collapse state indicated by the response history analysis conducted in the current investigation made it necessary to re-assess this limit. The analysis results indicated that in some cases the limits indicated in ASCE 41 [22] were exceeded, and yet in other cases the numerical instability was reached prior to developing these limits. Hence, the collapse was defined in the current investigation as the median of the maximum inter-storey drift ratios attained on the IDA curves.

5. Development of Fragility Relationships

The probability of drift demand (D) at a given intensity, $S_a(T_e)$, i.e., the spectral value at effective period, or $S_a(T_d)$, i.e., the spectral value at design period, was calculated with the method adopted by Cornell et al. [23]. The conditional median of drift demand, $D_{M,}$ was expressed as a power function, $D_M = a[S_a(T_e)]^b\varepsilon$ or $D_M = a[S_a(T_d)]^b\varepsilon$; where a and b were regression coefficients and ε was lognormal random variable [24]. It was assumed that the demand had lognormal probability distribution at a given spectral acceleration with the median lognormal random variable equal to unity ($\varepsilon = 1$). Logarithmic standard deviation of lognormal random variable ($\sigma_{\ln\varepsilon}$) was equal to the standard deviation of log of demand (σ_D) [25]. The regression coefficient of the power function was calculated by linear regression in logarithmic space of the 'cloud' response using least square method. The standard deviation of log of demand (σ_D) was assumed constant with variation of spectral acceleration, $S_a(T_e)$ or $S_a(T_d)$. The dispersion for all limit states (σ_{LS}) was considered as 0.3 [26] and the uncertainty in analytical modeling (σ_M) was taken as 0.2 with 90% confidence that the analytical model findings were within 30% of actual value [21]. The effects of aleatoric and epistemic uncertainty were calculated according to the equation suggested by Zareian and Krawinkler [27], as shown below:

$$\sigma_{EQU} = \sqrt{\sigma_{LS}^2 + \sigma_M^2} \tag{1}$$

where, σ_{EQU} is the uncertainty component associated with aleatoric and epistemic effect in demand estimation, which was found to be 0.36 in this study. The total uncertainty in finding the probability of collapse, σ_{TOT}, is:

$$\sigma_{TOT} = \sqrt{\sigma_{EQU}^2 + \sigma_D^2} \tag{2}$$

The above computed parameters are then substituted into the equation shown below to find the conditional probability of exceeding a limit state at a given intensity, $S_a(T_e)$ or $S_a(T_d)$.

$$P_{LS} = 1 - \Phi\left(\frac{lnD_c - lnD_m}{\sigma_{TOT}}\right) \tag{3}$$

where, D_C is median drift capacity specified for a limit state. The fragility curves are then presented as plots of P_{LS} versus $S_a(T_e)$ or $S_a(T_d)$. They are shown in Figures 4 and 5 for 2, 5, and 10-storey frame buildings located in Ottawa and Vancouver.

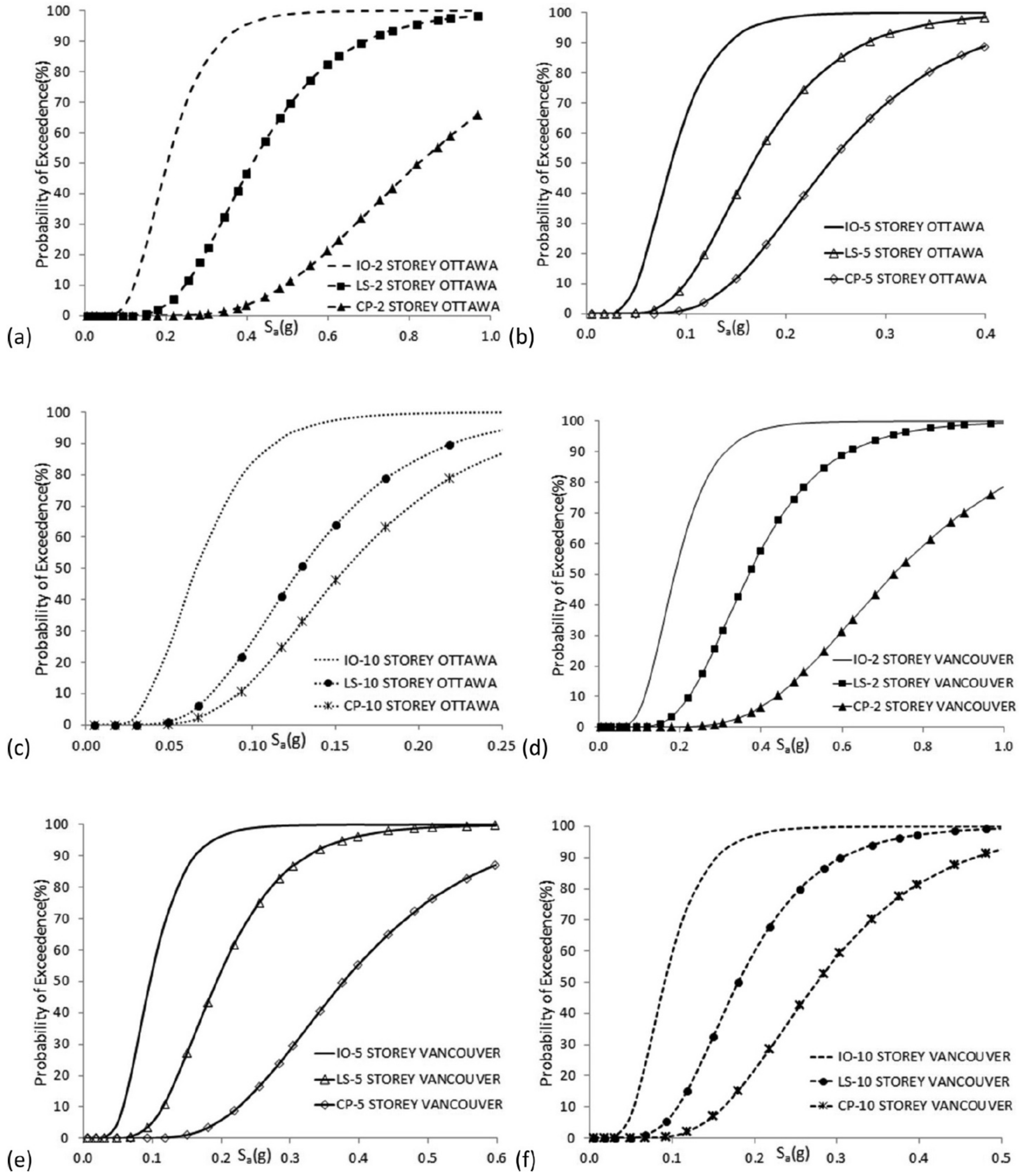


Fig. 4 – Fragility curves for (a) 2-storey, (b) 50storey, (c) 10-storey buildings in Ottawa and (d) 2-storey, (e) 5-storey, (f) 10-storey buildings in Vancouver.

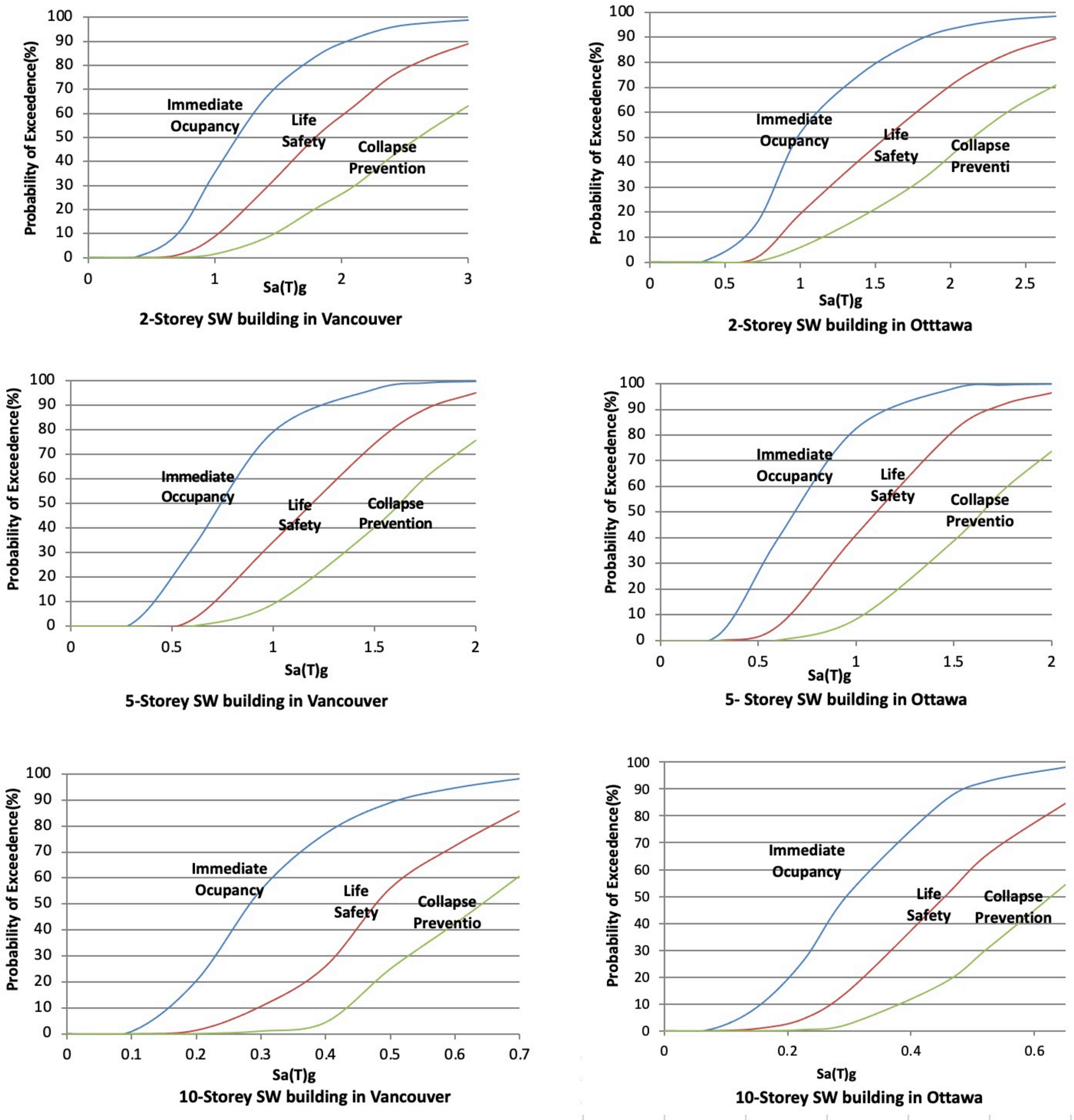


Fig. 5 – Fragility curves for shear wall buildings in Vancouver and Ottawa

6. Observations and Conclusions

The fragility analysis conducted in this investigation indicate the following:

- At design period spectral acceleration, the Ottawa frame buildings showed less probability of exceeding limit state performance levels when compared with those for Vancouver. The frame buildings in Vancouver, designed after the 1985 threshold year showed on average 43% probability of exceeding the NBC target performance level of life safety at design earthquake, whereas the same performance level is exceeded with an average probability of 0% in frame buildings located in Ottawa.

- The frame buildings in Vancouver showed higher inter-storey drift at collapse, with 6% probability of exceedance than those in Ottawa, which showed no probability of exceedance.
- The fragility curves based on $S_a(T_d)$ did not show a significant difference in probabilities of exceeding the CP performance levels when compared with those developed based on $S_a(T_e)$. Because the design period of T_d reflects the as-built conditions of the buildings incorporating the possible stiffening effects of non-structural elements, it may be more appropriate to use them for seismic vulnerability assessment, with the fragility curves based on T_e reflecting possible softening of buildings during response.
- The results based on the empirical code periods indicate that 2-storey and 5-storey shear wall buildings recently designed in Vancouver meet the life safety performance limit, exhibiting 5% and 10% probability of exceedance at the NBC 2010 hazard levels.
- The 10-storey shear wall building designed for Vancouver has higher probability of life safety exceedance at the spectral acceleration corresponding to empirical code period, showing 50% probability of exceedance. For the same intensity level, the 2, 5 and 10-storey shear wall buildings showed 25%, 50% and 85% probability of exceeding the immediate occupancy performance limit, respectively.
- The 2-storey and 5-storey shear wall buildings in Ottawa indicated 0% probability of exceeding life safety, while the 10-storey building showed 3% of probability at code spectral acceleration corresponding to the empirical period. The probabilities of exceeding immediate occupancy limit state for the same three shear wall buildings in Ottawa are 10%, 5% and 25%, respectively.
- The fragility curves provided in this paper can be used to provide approximate seismic risk assessment for buildings designed after the 1985 design practice.

7. References

References must be cited in the text in square brackets [1, 2], numbered according to the order in which they appear in the text, and listed at the end of the manuscript in a section called References, in the following format:

[1] Geological Survey Canada (GSC) (2016): Earthquake zones in Canada, available from http://www.earthquakescanada.nrcan.gc.ca/zones/index-en.php [accessed 12 May 2016].

[2] National building code of Canada (NBC) (1941): National Research Council of Canada. Ottawa, Ont., Canada.

[3] National building code of Canada (NBC) (1953): National Research Council of Canada. Ottawa, Ont., Canada.

[4] National building code of Canada (NBC) (1965): National Research Council of Canada. Ottawa, Ont., Canada.

[5] National building code of Canada (NBC) (1970): National Research Council of Canada. Ottawa, Ont., Canada.

[6] National building code of Canada (NBC) (1975): National Research Council of Canada. Ottawa, Ont., Canada.

[7] National building code of Canada (NBC) (1980): National Research Council of Canada. Ottawa, Ont., Canada.

[8] National building code of Canada (NBC) (1985): National Research Council of Canada. Ottawa, Ont., Canada.

[9] National building code of Canada (NBC) (1995): National Research Council of Canada. Ottawa, Ont., Canada.

[10] National building code of Canada (NBC) (2005): National Research Council of Canada. Ottawa, Ont., Canada.

[11] National building code of Canada (NBC) (2010): National Research Council of Canada. Ottawa, Ont., Canada.

[12] National building code of Canada (NBC) (2015): National Research Council of Canada. Ottawa, Ont., Canada.

[13] Canadian Standards Association (CSA) (1973): Design of concrete structures. CSA. CSA A23.3-1973. Mississauga, Ont., Canada.

[14] Canadian Standards Association (CSA) (1984): Design of concrete structures. CSA. CSA A23.3-1984. Mississauga, Ont., Canada.

[15] National Research Council of Canada (NRCC) (1993): Manual for screening of buildings for seismic investigation. ISBN 0-660-15381-5. NRCC 36943. Ottawa, Ont., Canada.

[16] Vamvatsikos, D., and Cornell, C.A. (2002): Incremental dynamic analysis. Earthquake Engineering & Structural Dynamics, 31(3): 491–514. doi:10.1002/eqe.141.

[17] Canadian Standards Association (CSA) (2004): Design of concrete structures. CSA. CSA A23.3-2004. Mississauga, Ont., Canada.

[18] Vamvatsikos, D., and Cornell, C.A. (2004): Applied incremental dynamic analysis. Earthquake Spectra, 20(2): 523–553.

[19] Computers and Structures, Inc. (CSI) (2013): PERFORM-3D. Version 5.0.1. Computers and Structures, Inc. Berkeley, CA, United States.

[20] Atkinson, G.M. (2009): Earthquake time histories compatible with the 2005 National building code of Canada uniform hazard spectrum. Canadian Journal of Civil Engineering, 36(6): 991–1000.

[21] Ellingwood, B.R., Celik, O.C., and Kinali, K. (2007): Fragility assessment of building structural systems in Mid-America. Earthquake Engineering & Structural Dynamics, 36(13): 1935–1952.

[22] American Society of Civil Engineers (ASCE). 2007. Seismic evaluation and retrofit of existing buildings. ASCE/SEI 41-13. Va., United States.

[23] Cornell, CA., Jalayer, F., Hamburger, R.O., and Foutch, D.A. (2002). Probabilistic basis for 2000 SAC Federal Emergency Management Agency steel moment frame guidelines. Journal of Structural Engineering, 128 (Special Issue: Steel moment frames after Northridge-PartII): 526–533.

[24] Ramamoorthy, S., Gardoni, P., and Bracc, J. (2006): Probabilistic demand models and fragility curves for reinforced concrete frames. Journal of Structural Engineering, ASCE, 132(10): 1563–1572.

[25] Jeong, S., Mwafy, A.M., and Elnashai, A.S. (2012): Probabilistic seismic performance assessment of code-compliant multi-story RC buildings. Engineering Structures, 34: 527–537.

[26] Jeong, S.H., and Elnashai, A.S. (2007): Probabilistic fragility analysis parameterized by fundamental response quantities. Engineering Structures, 29(6): 11238–1251.

[27] Zareian, F., and Krawinkler, H. (2007): Assessment of probability of collapse and design for collapse safety. Earthquake Engineering and Structural Dynamics, 36(13): 1901–1914.

