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Seismic Fragility Curves for Reinforced Concrete Frame Buildings in Canada Designed After 1985

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Abstract: Performance of reinforced concrete frame buildings depends on seismic hazard of the region and vulnerability of the structure. Performance-based evaluation of buildings may be conducted through fragility curves developed for different levels of performance. Reinforced concrete frame buildings with 2, 5 and 10-stories, designed as moderately ductile buildings for Ottawa in Eastern Canada and fully ductile buildings for Vancouver in Western Canada were used to generate seismic fragility curves. The buildings were analyzed using PERFORM-3D software to assess seismic vulnerabilities. Incremental Dynamic Analysis was employed to generate fragility curves. Two sets of earthquake records compatible with Uniform Hazard Spectra of 2010 NBCC were selected, where each set contained 20 records for each city. The fragility curves depict probabilities of exceedances for different damage states, and can be used for seismic vulnerability assessment of reinforced concrete frame buildings in Canada designed and built after 1985.

Keywords: Fragility analysis; incremental dynamic analysis; performance-based evaluation; reinforced concrete frame buildings; seismic assessment; seismic damage estimation; vulnerability analysis.

1. Introduction

Earthquakes occur frequently in Canada with historic damage. The 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 about three magnitude 5 earthquakes taking place in each decade (GSC 2016). 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. Eastern earthquakes tend to be less frequent and of moderate magnitude (GSC 2016). This difference in seismic regions is reflected in building design practices that follow the requirements of the National Building Code of Canada (NBCC).

It is preferable to conduct seismic vulnerability assessment 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 (Vamvatsikos and Cornell 2002). It is best suited to earthquake investigations, with capabilities for providing support to decision makers.

The objective of this paper is to present seismic fragility curves for reinforced concrete frame buildings in Canada. It forms part of a comprehensive research program currently underway at the University of Ottawa involving reinforced concrete frame and shear wall buildings, as well as masonry buildings, with or without irregularities, designed during different periods of building code development.

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 (1941 NBCC). Earlier equations defined seismic base shear as a percentage of seismic weight of building as seismic coefficient. In the 1953 NBCC, 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 NBCC, the differences in construction type and associated level of ductility was 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 NBCC 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 NBCC Commentary with ductility factors for different building types for use in dynamic analysis. The requirements remained essentially the same in the 1980 NBCC with refinements made to seismic response coefficient S as affected by fundamental period. New seismic zoning maps were introduced in the 1985 NBCC 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 (1990 NBCC), 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 (2005 NBCC) with new site specific uniform hazard spectra having 2% in 50 year probability of exceedance. The approach was kept the same in the 2010 NBCC with new hazard values introduced in the 2015 NBCC. The hazard values in the 2015 NBCC are 17% to 28% higher for Vancouver and 24% to 15% lower for Ottawa relative to those in the 2010 NBCC within the 0.5 sec to 1.0 sec building period range.

The design and detailing requirements for reinforcement concrete buildings in CSA A23.3 went through a similar evolution. There were no seismic design requirements prior to CSA A23.3-1973 (CSA 1973), which was referenced in the 1975 NBCC. Ductile design and detailing requirements for seismic resistance were introduced for the first time in 1973, which remained the same until 1984. Significant improvements were made to the standard in 1984 (CSA 1984) 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 NBCC, which made reference to

CSA A23.3-1984. 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 bench mark year in the Canadian Seismic Screening Manual (NRC 1992).

An extensive comparison of seismic demands and capacities of reinforced concrete frame buildings was performed as part of the current investigation to assess the significance of variations in building designs between 1985 and 2010. Static seismic base shears for the 2, 5 and 10-storey buildings used in the current investigation were calculated using Equivalent Static Force Procedure (ESFP). Figure 1 shows the evolution of seismic design base shear ratio (design base shear V / building weight used for base shear calculations W) assuming buildings in Vancouver have full ductility and those in Ottawa have moderate ductility. It was observed that the change in equivalent seismic base shear (based on the empirical code period) between 1985 and 2010 varied between 0.8 and 1.3 of the 1985 values for 10-storey and 2-storey moderately ductile buildings in Ottawa, respectively. The range was 1.2 and 1.8 of the 1985 values for 10-storey and 2-storey ductile buildings in Vancouver. However, this change in seismic shear demand did not translate into equivalent levels of change in member capacities in final building designs. The seismic load combinations changed from $1.25D+0.7(1.5L+1.5E)$ in 1985 to $1D+0.5L+1E$ starting in 1990, implying that the contribution of gravity loads to member design would be higher in 1985 and lower in the post 1990 codes. The changes in load factors may offset the effects of changes in seismic base shear on final designs. Where they don't, as in the case of the 10-storey building in Ottawa, they create higher design capacity with reduced seismic vulnerability. Furthermore, the concrete resistance factor, ϕ_c was 0.6 in CSA A23.3-1984, whereas it was increased to 0.65 in CSA A23.3-2004 (CSA 2004). This results in a nominal capacity that is closer to design resistance in newer buildings. The comparison implies that there is more reserve capacity in older buildings, even though the seismic design force levels at the time were generally lower. Therefore, the final structural designs of the three buildings

considered are not expected to show significant variations between 1985 and 2010. Hence, reinforced concrete frame buildings designed between 1985 and 2010 can be grouped together for the purpose of seismic vulnerability assessment within the ductility ranges considered in design.

2. Selection and Design of Buildings

Three regular frame buildings with 2-storey, 5-storey and 10-storey heights were selected for Ottawa, representing eastern Canadian seismicity, and for Vancouver, representing western Canadian seismicity. All the buildings consisted of moment resisting frames in both orthogonal directions with 5 bays in each direction, having a 7.0 m span length, resulting in a 35m by 35m square floor plan. Typical floor height of 4.0 m was used for each floor including the ground level. Figure 2 shows elevation view of the buildings. The design dead load included a superimposed dead load of 1.33 kPa consisting of floor finish, partition walls and mechanical/electrical fixtures, in addition to member self-weight. The live load was 2.4 kPa.

The buildings were designed based on the 2010 NBCC seismic requirements with the accompanying CSA Standard A23.3-04 "Design of Concrete Structures" 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 residential 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 through the use of SAP 2000 (CSI 2013a.) software with reduced section properties according to CSA A23.3-04. These dynamic fundamental periods were longer than those computed by the code-recommended empirical values by more than 150%. Therefore, the period values were taken as 1.5 times the values computed based on the empirical code equations for design. Uniform Hazard Spectra (UHS) values were used for design as prescribed in the 2010 NBCC. These corresponded to spectral accelerations (S_a) of 0.298g,

0.134g and 0.066g for 2-storey, 5-storey and 10-storey building in Ottawa and 0.618g, 0.320g and 0.204g for the buildings in Vancouver, respectively. The buildings in Ottawa consisted of moderately ductile frames designed with ductility related force modification factor R_d and over-strength related force modification factor R_o as 2.5 and 1.4, respectively. The buildings in Vancouver consisted of fully ductile frames, designed with $R_d = 4.0$ and $R_o = 1.7$. Concrete compressive strength, f'_c , was taken as 40 MPa for 2-storey and 10-storey buildings and 30 MPa for 5-storey building. Rebar with 400 MPa yield strength was used as reinforcement for all members. The buildings were analysed and designed by using software ETABS (CSI 2008) with the load cases defined in NBCC (2010). Reinforcement obtained from the software was used along with seismic detailing implemented manually for complete member design. Table 1 provides the design details for each member. The buildings in Vancouver and Ottawa had the same member dimensions because of similar ratios of seismic demand to inelastic capacity in each city. This resulted in the same fundamental period for the same height buildings. Accordingly, the design fundamental periods for the two, five and ten-storey buildings were computed to be 0.54 sec, 1.06 sec and 1.79 sec, respectively. The effective fundamental periods computed using the Eigen Value solution were 1.08 sec, 2.04 sec, and 2.84 sec for the two, five and ten storey buildings, respectively.

3. Incremental dynamic analysis (IDA)

The present study focuses on developing fragility response of reinforced concrete frame structures in Canada with regular structural layout through incremental dynamic analysis (IDA). A set of 20 earthquake records were selected and IDA was employed to generate fragility curves. IDA was conducted for each seismic record with incrementally varying intensity levels, resulting in an IDA curve providing a relationship between earthquake intensity and a structural deformation quantity. In the current investigation the maximum inter-storey drift ratio, Θ_{max} , was

used as a damage measure (DM) and 5% damped spectral acceleration was used as an intensity measure (IM) either at design period T_d or at effective period T_e obtained from dynamic analysis. Each earthquake record was scaled in such a way that the successive run would always be within 10% of the previous IM level. Hunt and fill algorithm was used to limit the number of runs while covering the entire range of structural performance (Vamvatsikos and Cornell 2004). Accordingly, the dynamic analysis was first conducted under a reduced earthquake intensity to correspond to a relatively low spectral acceleration of 0.005g to ensure elastic response. In the second analysis, when the spectral accelerations corresponded to T_e , the seismic record was amplified such that the increase in spectral increment was 0.05g with a step increment of 0.025g up to failure. The same approach was used for the IDA when seismic records were amplified based on $S_a(T_d)$, except for the increase in increments, which was twice the incremental increase used earlier for $S_a(T_e)$. The structural failure was defined either by side-sway collapse (structural instability) or when the rate of change in deformations (the slope of the IDA curve) reached 20% of the initial effective elastic slope as also defined in FEMA 350 (FEMA 2000a). Side-sway collapse was defined as the point of dynamic instability when inter-storey drift increased without bound (Goulet et al. 2007). Figure 3 illustrates the definition of maximum drift capacity used in the current study. IDA was used to develop fragility response for different performance levels with associated limit states.

4. Modelling for Dynamic Analysis

IDA was conducted using software PERFORM-3D (CSI 2013) to perform nonlinear dynamic analysis and evaluate the inelastic performance of structural components. PERFORM-3D is specialized software for damage assessment, and is used by other researchers (Reyes and Chopra 2012; Ghodsi and Ruiz 2010; Zeynep Tuna 2012; Wen-Cheng Liao 2010; Michael W. Hopper 2009). Three-dimensional analytical models of the bare frame structures designed earlier were

developed for dynamic analysis. The frame elements (beam and columns) were modeled to deform in double curvature with two symmetrical segments. Each segment consisted of an elastic beam element and a plastic hinge. Chord rotations were used to define the member end rotations. All beam and column elements had stiff end zones at the joints that represented the end portions built integrally with the adjoining members. The stiffness of these end zones was assigned a value equal to 10 times the member stiffness.

Element stiffness was specified as per the requirements of CSA A23.3-04. Both beam and column rigidities were reduced to account for concrete cracking; and effective inertia, I_e , were assigned to the members. Hysteretic behaviour of potential plastic hinge regions was modelled by assigning a stiffness degrading model in PERFORM-3D. The software uses perfectly elasto-plastic hysteretic relations, modified for stiffness and strength degradation under reversed cyclic loading as illustrated in Fig. 4. The stiffness degradation is introduced through the “energy degradation factor (EDF),” which is the ratio of the area under an elasto-plastic to stiffness degrading hysteresis loops. In the current investigation, EDF was computed from experimental observations. Tests of reinforced concrete elements conducted by Ozcebec and Saatcioglu (1987) were used for this purpose. It was found that well confined flexure-controlled elements showed behaviour that could be modelled with the use of $EDF = 0.62$ up to the yield point, and 0.56 thereafter. The same EDF values were used for both moderately ductile and fully ductile elements. The sensitivity analysis conducted by authors indicated that minor variations in EDF did not result in significant changes in dynamic response (Al Mamun and Saatcioglu 2016)¹.

The envelope curves for the hysteretic models were defined in terms of nominal moment resistances and corresponding chord rotations. This was done according to the ASCE 41-13 (ASCE 2014) guidelines. The yield moment (M_Y) and chord rotation at yield (Θ_Y) were

¹ Al Mamun and Saatcioglu. 2016. Seismic performance evaluation of RC frame structures using PERFORM-3D. Submitted for publication in *Earthquake Spectra*, the Journal of EERI.

calculated for each element from sectional analysis. The post yield stiffness was defined as strain hardening stiffness with 3% and 4% of the effective elastic stiffness for beam and column elements, respectively up to ultimate capacity (M_U). The ultimate capacity depended on the ductility ratios adopted for moderately ductile and fully ductile structures. The ductility related force modification factor (R_d) values, specified in the 2010 NBCC were used as 2.5 and 4.0 for moderately ductile and fully ductile buildings. These ductility ratios were also reported to have been observed during previous column tests (Park et al. 1982; Beng Ghee et al. 1981; Zahn et al. 1986; Saatcioglu and Ozcebec 1989), though some researchers showed that well confined concrete column could achieve ductility ratios higher than 4.0 irrespective of the level of axial compression. The ultimate rotational capacity (Θ_U) was defined in the current investigation as 2.5 and 4.0 times the yield rotation (Θ_Y) for nominally and fully ductile elements as the onset of strength decay points. The degradation slope of moment-rotation envelope was computed to be the same as that recommended by ASCE 41-13. The ASCE 41-13 ultimate plastic chord rotation (Θ_U) and the residual moment capacity (M_R) depend on the level of axial compression and the confinement steel area ratio. The linear descending branch of the envelop curve continued until the residual moment capacity (M_R) as defined in ASCE41-13 as a ratio of the ultimate moment capacity (M_U). This ratio for the beams was 0.2 and varied for columns between 0.2 and 0 as a function of axial load. Fig. 5 shows the details of moment-rotation envelope for members having the yield capacity, but different levels of ductility. Further details of analytical modelling techniques employed are reported elsewhere (Al Mamun and Saatcioglu 2016).

The members were modelled to behave elastically in shear. This is consistent with CSA A23.3-04, which requires higher shear capacity than that corresponding to flexural capacity to prevent brittle shear failure while promoting ductile flexural response. This is a preferred performance observed by researchers (Priestley et al. 1994). Ozcebec and Saatcioglu (1989) experimentally observed that deflections due to shear in flexure-dominant members accounted

for 22% of the total deflection, even though local shear deformations within the plastic hinge could be as high as 83% of the hinging region deformations. The contribution of shear to total member deflection was observed to decrease (forming 8% of total deflection in one column test) as inelastic deformations increased in flexure (Saatcioglu and Ozcebec 1989). Linear elastic shear properties of structural elements were also used by previous researches (Borzi et al. 2008; Inel and Ozmen 2006; Liel et al. 2011).

5. Selection of Earthquake Records

Synthetic earthquake records, developed for Ottawa and Vancouver, with 2% probability of exceedance in 50 years were selected for the development of the fragility curves. These records were compatible with the Uniform Hazard Spectra (UHS) specified in NBCC (2010), and were developed by Atkinson (2009). The records were modified as suggested by Atkinson (2009) to match the UHS for the period range of interest. The design period of the buildings considered in the current investigation varied between 0.5 and 2.0 seconds, and this range was used to modify the records. A set of twenty records was selected for buildings in Ottawa, and another set of twenty records was selected for buildings in Vancouver. Each set of records reflected two different magnitudes and two different distances, resulting in four different magnitude-distance combinations. Five records were selected from each magnitude level and distance category. For Ottawa, M6 earthquakes were selected with epicentral distances of 10-15 km and 20-30 km; and M7 earthquakes were selected with epicentral distances of 15-25km and 50-100 km. The duration of records were 5, 7, 17 and 20 seconds, respectively. For Vancouver, M6.5 earthquake was selected with epicentral distances of 10-15 km and 20-30 km; and M7.5 earthquake was selected with epicentral distances of 15-25 km and 50-100 km. The duration of records was 10, 15, 65 and 57 seconds, respectively. Acceleration response spectra were generated for 5% of

critical damping, and for Soil Type C. Figure 6 shows the comparison of response spectra for mean seismic records with the UHS of NBCC (2010) for Ottawa and Vancouver.

Fragility analysis requires IDA under different intensity of earthquakes. This necessitates the amplification of seismic records. Though it is common to amplify seismic records based on the spectral acceleration at fundamental period (Vamvatsikos and Cornell 2002), researchers used several different approaches to amplify seismic records. Jeong et al. (2012) used effective fundamental period based on pushover analysis. Kircil and Polat (2006) used elastic fundamental period as the reference point. In the present study seismic records were scaled to match the target spectral values obtained by the hunt and fill algorithm discussed earlier to represent different earthquake intensities. This scaling was done using two spectral values as the basis, first using the spectral acceleration that corresponded to the code defined design period (T_d), and secondly using the value corresponded to the effective period computed by dynamic analysis using cracked (effective) moment of inertia, T_e . The use of two sets of scaling resulted in two sets of fragility curves as discussed in the following paragraphs.

For each target spectral acceleration, seismic record was multiplied by a factor equal to $S_{a,Target}/S_a(T_d)$ or $S_{a,Target}/S_a(T_e)$, where $S_{a,Target}$ is the target spectral acceleration, $S_a(T_d)$ is spectral acceleration at design period (T_d) and $S_a(T_e)$ is the spectral acceleration at effective period (T_e). The amplification procedure was validated against the spectra of scaled records where the spectra were computed using software PRISM (2011). In all cases the amplified record was able to generate spectral values that matched $S_{a,Target}$. This is shown Fig. 7. The scaled records were then used to perform incremental dynamic analysis (IDA).

6. Limit States

The fragility curves were developed for different levels of performance. Commonly accepted performance levels were selected (ASCE 41 2013; FEMA 356 2000b; ACI 374.2R-13 2013). They consisted of; i) Immediate Occupancy (IO), ii) Life Safety (LS), and iii) Collapse Prevention (CP). Inter-storey drift ratio was used as a damage indicator, defining the limit state for each performance level. The inter-storey drift limits were adopted from previous standards/recommendations as 1% and 2% for IO and LS performance levels (ASCE 41-13, FEMA 356, ACI 374.2R-13). The CP performance limit state depended on the onset of strength decay, which in turn depended on the ductility capacity of structural elements. Jeong et al. (2012) used FEMA 356 limit of 4% inter-storey drift, Akkar et al. (2005) used 75% of the median of maximum inter-storey drifts from the records considered, Erberik (2008) used 75% of the mean of maximum inter-storey drifts, Kircil and Polat (2006) used 5% probability of attaining collapse with 95% confidence level, and Ellingwood et al. (2007) used the median of maximum inter-storey drift ratio. In the current investigation CP limit state was defined as the median of the maximum inter-storey drift ratio attained on the IDA curve.

The IO limit represents very limited structural damage, where the force resisting system nearly retains the pre-earthquake strength and stiffness. Since the risk of fatal injury is very low, the building can be reoccupied immediately. Various approaches were used by researchers to identify the IO limit state drift. Jeong et al (2012) used the inter-storey drift corresponding to the first yield of a structural member, Akkar et al. (2005) used the global yield drift ratio, and Kircil and Polat (2006) used the maximum inter-storey drift ratio at 5% probability of yielding with 95% confidence level. Erberik (2008) used softening index (SI) proposed by DiPasquale and Cakmak (1987) as serviceability limit state, analogous to IO. SI was defined as:

$$(1) \quad SI = 1 - \frac{T_e}{T_j}$$

where T_j is the effective period at intermediate spectral acceleration. $SI = 0.20$ was attained at IO limit state when $T_j = 1.25 T_e$. This measure of performance was believed to be more reliable than using 1% drift, since SI provided inter-storey drift ratio for the IO performance level corresponding to seismic records.

7. Development of Fragility Relationships

The probability of drift demand (D) at a given Intensity, $S_a(T_e)$ or $S_a(T_d)$, was calculated with the method adopted by Cornell et al. (2002). 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 (Ramamoorthy et al. 2006). 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) (Jeong et al. 2012). 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 value of regression coefficients a and b and standard deviation of log of demand (σ_D) are shown in Fig. 8 for Ottawa and Vancouver buildings for which the analyses were performed with scaled seismic records based on $S_a(T_e)$ or $S_a(T_d)$. The dispersion for all limit states (σ_{LS}) was considered as 0.3 (Jeong and Elnashai 2007) 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 (Ellingwood et al. 2007). The effects of aleatoric and epistemic uncertainty were calculated according to the equation suggested by Zareian and Krawinkler (2007), as shown below:

$$(2) \quad \sigma_{\text{EQU}} = \sqrt{\sigma_{\text{LS}}^2 + \sigma_{\text{M}}^2}$$

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} , was:

$$(3) \quad \sigma_{\text{TOT}} = \sqrt{\sigma_{\text{EQU}}^2 + \sigma_{\text{D}}^2}$$

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 $S_a(T_d)$.

$$(4) \quad P_{\text{LS}} = 1 - \Phi\left(\frac{\ln D_c - \ln D_M}{\sigma_{\text{TOT}}}\right)$$

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 Fig. 9 and Fig. 10 for 2, 5, and 10-storey frame buildings located in Ottawa and Vancouver.

8. Seismic Performance Evaluation

The results of the IDA analysis indicated different levels of inelasticity and sequence of hinging among frame members at different performance levels. The yielding of beams was observed prior to developing the IO performance level. The beam yielding occurred at lower floors first, followed by the yielding of the first-storey columns. Upon the yielding of the columns, inter-storey drift levels increased considerably as buildings approached the LS performance level. At LS performance level, more hinging was observed in buildings located in Ottawa than those in Vancouver. Almost all the first-storey columns hinged at LS level, with increased plastic hinging in beams at upper floors.

The CP performance level was associated with significant plastification of members, followed by the failure of individual elements. In the 2-storey buildings, initial hinges formed in the 2nd storey beams, followed by the 1st storey columns. The structure reached collapse due to the failure of the 2nd storey beams, all in the direction of seismic loading, followed by the 1st storey columns. In the 5-storey buildings, hinges formed in the exterior and interior beams at the 3rd storey level, which extended to the 4th, 5th and 2nd floors. Column hinging occurred at the 1st storey level, followed by the columns at the 3rd floor. Because the columns of the first 2 floors had higher capacities, the 2nd floor column hinging followed the hinging of columns at the 3rd to 4th floors. The structure reached collapse level performance due to the failure of the first-storey columns at the base, following the failures of beams at the 3rd, 4th, 5th and 2nd floor levels. In the 10-storey buildings, the hinges formed in the 2nd floor interior beams, and extended to the 8th floor interior beams, followed by the exterior beams. Subsequently, the 1st storey columns hinged at their base, followed by the hinging of the columns at the 3rd floor. The same building under high intensity earthquakes showed failure of both interior and exterior beams between the 2nd and 7th floors, followed by the failure of the 3rd storey columns.

The yield rotations (Θ_Y) in members were computed to be similar for the same height buildings in Ottawa and Vancouver. However, the member ductility was different depending on design ductility demands. The buildings in Ottawa were designed and detailed for ultimate chord rotations of $\Theta_U = 2.5\Theta_Y$, whereas those in Vancouver were designed and detailed for $\Theta_U = 4.0\Theta_Y$. This implies that the onset of strength decay in members of the Ottawa buildings started at lower rotational values in comparison to those in Vancouver. Therefore, the maximum inter-storey drift of the Ottawa buildings at the CP performance level were lower than those for Vancouver buildings. This is shown in Table 2 for both sets of analyses based on the two approaches used for scaling, i.e., $S_a(T_e)$ and $S_a(T_d)$.

The fragility response of structures shown in Fig. 9 and Fig. 10 indicate a similar trend among the buildings in Ottawa and Vancouver, i.e., 10-storey buildings reached the CP level of performance at lower levels of spectral accelerations because these buildings, with longer periods, were designed for lower spectral accelerations. The 2-storey buildings showed CP level of performance at higher spectral values. Limit state probabilities at effective period and design period are given in Table 2 for two sets of fragility analysis, developed either with $S_a(T_e)$ established based on the building periods obtained from dynamic analysis, which incorporate the effect of structural softening and period elongation during response, or $S_a(T_d)$ established based on the NBCC-recommended building periods. The fragility curves developed based on $S_a(T_e)$ indicated that, on average, the probability of exceeding the IO performance level was 17% for buildings in Ottawa at the design earthquake intensity, whereas the buildings in Vancouver had 93% probability of exceeding the same level of performance. At the same intensity, the buildings in Ottawa developed no probability of exceeding the LS performance level, while those in Vancouver showed 43% of exceeding the same level of performance. The probability of exceeding the CP performance level at design intensity in Ottawa was 0%, whereas in Vancouver it was 6%. The fragility curves depict a trend of increasing probabilities of exceeding performance limits with increasing number of stories; except for 5-storey building in Ottawa at IO performance level, which was slightly lower than that for 2-storey building. Table 1 also shows inter-storey drift ratios ($\Theta_{\max}$) at the CP performance level. It is noteworthy that the margin between the LS and CP performance levels is higher for buildings in Vancouver, relative to those in Ottawa. Similar response of building performance was obtained from the fragility curves for $S_a(T_d)$; buildings in Ottawa had less probability of exceeding limit states in comparison with buildings in Vancouver. The comparison of building performance as obtained from the fragility curves for $S_a(T_e)$ and $S_a(T_d)$ indicate similar probabilities of exceeding LS performance level in Ottawa. The same is true for the CP performance level for cities. The code

specified T_d incorporates potential stiffening effects of non-structural members that may be present in the building. In reality, the structural period falls between the code specified value of T_d and the dynamic period computed based on a model that consists of structural elements only (T_e). However, the probabilities obtained from the fragility curves developed on the basis of $S_a(T_e)$ and $S_a(T_d)$ yield similar probabilities of exceedance for the collapse performance level.

To observe the effect of ductility, additional fragility curves were developed for 2-storey and 5-storey frame buildings in Ottawa when the buildings were designed for the same seismic force level associated with moderate ductility but having full ductility (ductility=4.0). This resulted in higher inter-storey drift ratios at the CP performance level. This set of curves is shown in Fig. 11, indicating the same probabilities for the IO and LS performance levels, but lower probabilities of exceedance at the CP performance level as compared to those for the moderately ductile buildings.

The effect of using softening index (SI) on inter-storey drift ratio for IO performance level was also investigated. The results showed that, using SI resulted in an average of 0.40% and 1.25% inter-storey drift ratios for buildings in Ottawa and Vancouver, respectively, in comparison with the 1% drift ratio recommended by the existing standards (ASCE 41-13, FEMA 356, ACI 374.2R-13). This observation implies that the Ottawa structures yielded at a lower rotation than those in Vancouver.

8. Summary and Conclusions

Fragility curves were developed for 2-storey, 5-storey and 10-storey buildings in Ottawa and Vancouver for vulnerability assessment of reinforced concrete frame buildings with regular structural layouts. The earthquake records were scaled based on spectral accelerations at effective period (T_e) and design period (T_d) of structures. It was found that at design period

spectral acceleration, the Ottawa buildings showed less probability of exceeding limit state performance levels when compared with those for Vancouver. The Buildings in Vancouver, designed after the 1985 threshold year showed on average 43% probability of exceeding the NBCC target performance level of life safety at design earthquake, whereas the same performance level is exceeded with an average probability of 0% in buildings located in Ottawa. The 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 fragility curves developed in this investigation were all generated using buildings with specific heights, designed based on the 2010 NBCC using moderately ductile and fully ductile design requirements for Ottawa and Vancouver, respectively. While the same ductility capacities may be attained in buildings designed between 1985 and 2010, the design force levels may show some fluctuations during this period. Therefore, the use the same fragility curves for buildings designed by editions of NBCC other than the 2010 version requires judgement with respect to the effects of variations in design force levels.

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Table 1. Structural member details of 2, 5, 10-storey structures.

	2 Storey		5 Storey		10 Storey		
	East and West		East and West			East	West
	Size	Rebar	Size	Rebar	Size	Rebar	Rebar
Corner Column 1-Top	300X300	8-20M	300X300	8-20M	350X350	4-25M+4-15M	4-25M+4-15M
Ext Column 1-2	300X300	4-30M	300X300	4-30M	350X350	16-20M	4-30M+8-20M
Ext Column 3-Top	-	-	300X300	4-25M	350X350	4-25M+4-15M	8-25M
Int-1 Column 1-2	325X325	4-20M+4-15M	450X450	12-25M	500X500	12-30M+6-20M	12-30M+6-20M
Int-1 Column 3-5	-	-	450X450	4-25M+4-20M	500X500	8-25M+8-20M	8-30M+8-25M
Int-1 Column 6-10	-	-		-	500X500	4-25M+4-15M	4-25M+4-20M
Int-2 Column 1-2	325X325	4-20M+4-15M	450X450	4-30M+8-20M	500X500	12-30M+6-20M	12-30M+6-20M
Int-2 Column 3-5	-	-	450X450	4-30M	500X500	8-25M+8-15M	12-25M+8-20M
Int-2 Column 6-10	-	-		-	500X500	4-25M+4-15M	4-25M+4-20M
Int-3 Column 1-2	325X325	4-20M+4-15M	450X450	4-25M+8-20M	500X500	8-30M+8-25M	12-30M+6-20M
Int-3 Column 3-5	-	-	450X450	8-20M	500X500	4-25M+8-20M	12-25M+4-20M
Int-3 Column 6-10	-	-		-	500X500	4-25M+4-15M	4-25M+4-20M
Ext Beam Top	300X500	3-20M	300X500	3-20M	350X500	3-20M	3-20M
Ext Beam Bottom	300X500	2-20M	300X500	2-20M	350X500	2-20M	2-20M
		East	West				
Int Beam 1-5 Top	300X500	4-20M	4-20M	300X500	3-25M	350X500	4-25M
Int Beam 1-5 Bottom	300X500	3-20M	4-20M	300X500	2-25M	350X500	4-25M
Int Beam 6-10 Top	-	-	-	-	-	350X500	3-25M
Int Beam 6-10 Bottom	-	-	-	-	-	350X500	3-20M

Table 2. Comparison of limit state probabilities of 2, 5, 10-storey structures.

Building Height	City	T _d , sec	T _e , sec	S _a (T _d), g	S _a (T _e), g	Θ _{max} (%)	Fragility Based on S _a (T _d)			Fragility Based on S _a (T _e)		
							Prob. of Exceedance at S _a (T _d)			Prob. of Exceedance at S _a (T _e)		
							IO (%)	LS (%)	CP (%)	IO (%)	LS (%)	CP (%)
10-Storey	Ottawa	1.79	2.84	0.07	0.05	2.44	68	2	0	26	1	0
5-Storey	Ottawa	1.06	2.04	0.13	0.05	2.94	58	0	0	10	0	0
2-Storey	Ottawa	0.54	1.08	0.30	0.13	3.92	47	2	0	14	0	0
10-Storey	Vancouver	1.79	2.84	0.20	0.17	3.07	52	15	7	95	48	14
5-Storey	Vancouver	1.06	2.04	0.32	0.17	3.91	67	15	2	93	42	3
2-Storey	Vancouver	0.54	1.08	0.62	0.32	3.99	87	36	9	92	38	2

Fig. 1. Evolution of seismic design base shear ratio of 2, 5, 10-storey structures

Fig. 2. Elevation view of the buildings

Fig. 3. Maximum drift capacity on IDA Curve

Fig. 4. Effect of EDF on a Moment vs Total Chord Rotation hysteresis loop area in PERFORM-3D

Fig. 5. Typical moment-rotation envelope curve for same yield capacity member

Fig. 6. Comparison of mean spectral acceleration of seismic records with NBCC (2010) UHS for (a) Eastern Canada (Ottawa) and (b) Western Canada (Vancouver)

Fig. 7. Spectral acceleration for single record amplified based on $S_a(T_d)$

Fig. 8. Regression analysis of 2, 5, 10-storey structures in (a) Eastern Canada (Ottawa) and (b) Western Canada (Vancouver) with amplified seismic records based on $S_a(T_e)$; (c) Eastern Canada (Ottawa) and (d) Western Canada (Vancouver) with amplified seismic records based on $S_a(T_d)$

Fig. 9. Fragility response of (a) 2-storey, (b) 5-storey, (c) 10-storey structures in Eastern Canada (Ottawa) and (d) 2-storey, (e) 5-storey, (f) 10-storey structures in Western Canada (Vancouver) developed with amplified seismic records based on $S_a(T_e)$.

Fig. 10. Fragility response of (a) 2-storey, (b) 5-storey, (c) 10-storey structures in Eastern Canada (Ottawa) and (d) 2-storey, (e) 5-storey, (f) 10-storey structures in Western Canada (Vancouver) developed with amplified seismic records based on $S_a(T_d)$.

Fig. 11. Comparison of (a) 5-storey and (b) 2-storey structures fragility responses developed with Eastern Canada (Ottawa) seismic records.

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