Canadian Journal of Civil Engineering
 Revue canadienne de génie civil

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Journal:	<i>Canadian Journal of Civil Engineering</i>
Manuscript ID	cjce-2017-0238.R2
Manuscript Type:	Article
Date Submitted by the Author:	10-Feb-2018
Complete List of Authors:	Al Mamun, Abdullah; University of Ottawa, Civil Engineering Saatcioglu, Murat; Dept. of Civil Engineering,
Is the invited manuscript for consideration in a Special Issue? :	Not Applicable (Regular Submission)
Keyword:	Conventional concrete buildings, ductile concrete buildings, moderately ductile concrete buildings, seismic fragility curves, seismic vulnerability assessment

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**Seismic Fragility Analysis of Pre-1975 Conventional
Concrete Frame Buildings in Canada**

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Word Count: 8029

Abstract: Fragility analysis was conducted for reinforced concrete frame buildings in Canada designed based on the 1965 National Building Code of Canada as representative of pre-1975 era of seismic design practice. 2-, 5- and 10-storey buildings were designed for Vancouver and Ottawa, representing buildings in high and medium seismic regions. They were modelled for inelastic response time history analysis, with respective inelastic hysteretic models for flexure and shear. Software PERFORM-3D was used to conduct incremental dynamic analysis under incrementally increasing earthquake intensity. Probabilistic analysis of the results of incremental dynamic analysis led to the development of fragility functions, which can be used as seismic vulnerability assessment tools. The results are compared with those generated for frame buildings designed on the basis of the 2010 NBCC. The comparison indicates that the probabilities of exceeding performance levels are significantly higher for older buildings relative to recently built fully ductile and moderately ductile buildings, respectively.

Keywords: Conventional concrete buildings, ductile concrete buildings, moderately ductile concrete buildings, NBCC, reinforced concrete frame buildings, seismic design, seismic fragility curves, seismic performance, seismic vulnerability assessment.

39 1. Introduction

40 Seismic design requirements in the National Building Code of Canada (NBCC) have evolved
41 over the years. The procedure for determining seismic shear forces improved since its inception
42 in 1941. The effects of dynamic characteristics of structures, ductility demands for energy
43 dissipation, and improved seismic hazard values were implemented as the knowledge on
44 earthquake engineering has advanced. Older buildings designed prior to the enactment of modern
45 seismic codes remain vulnerable to earthquakes. The current research project addresses seismic
46 assessment of reinforced concrete frame buildings designed and built prior to the implementation
47 of ductile design provisions using a probabilistic approach. Fragility curves, relating seismic
48 hazard to probability of exceeding pre-selected performance levels, have been developed for
49 representative concrete frame buildings designed by the 1965 NBCC (NRCC 1965). The fragility
50 curves and the results of seismic vulnerability assessment are presented for Vancouver, located
51 in a high seismic region; and Ottawa, located in a moderate seismic region. The results are
52 compared with earlier fragility analysis conducted by the authors (Al Mamun and Saatcioglu
53 2017) for ductile and moderately ductile buildings in Canada conforming to the more recent
54 2010 NBCC (NRCC 2010) seismic provisions.

55 Seismic fragility curves provide a convenient procedure to assess vulnerability of structures
56 against seismic hazards. Though limited research was conducted in Canada on seismic fragility
57 analysis, Ventura et al. (2005) developed fragility curves for southwest British Columbia
58 involving different categories of buildings. Similar research was also conducted in the US,
59 Europe and Asia for employing fragility analysis for seismic risk assessment (Lupoi et al 2006;
60 Ramamoorthy et al. 2008; Buratti et al. 2010; Rajeev and Tesfamariam 2012; Bilgin 2013; Jeong
61 et al. 2007).

The 1941 National Building Code (NRCC 1941) defined base shear as percentage of building mass, which varied with the bearing capacity of soil. The 1953 NBCC (NRCC 1953) was the first edition where seismic hazard was expressed in terms of seismic zones, and a zoning map was used to calculate the base shear. In the 1965 NBCC, ultimate strength design was permitted as an alternative method and capacity reduction factors were introduced. Importance factor and foundation factor were also introduced as improvements that affected seismic base shear. In the 1970 NBCC (NRCC 1970), the base shear equation was modified to accommodate the effect of fundamental period. The seismic zoning map was revised. The advantage of structural ductility was recognized for the first time and a new factor was introduced to account for the ductility of structural system as affected by construction type. In the 1975 NBCC (NRCC 1975), the base shear equation was revised and dynamic analysis was permitted as an alternative method. In 1977 (NRCC 1977), the base shear computed by dynamic analysis was limited not to be less than the 90% of the value computed using the equivalent static load procedure. The requirements remained the same in the 1980 NBCC (NRCC 1980) with a small change in calculating seismic response factor, S . New seismic zoning maps were introduced in the 1985 NBCC (NRCC 1985). The base shear computed through dynamic analysis was limited not to be less than 100% of the value computed by the equivalent static load procedure. In the 1990 NBCC (NRCC 1990), force modification factor based on ductility demand was considered in computing base shear with revised equations for fundamental periods of frame buildings introduced in 1995 (NRCC 1995). Major revisions were introduced to the NBCC in the 2005 edition (NRCC 2005). New hazard values were introduced in the form of site-specific uniform hazard spectra (UHS). The equation for fundamental period of frame structures was revised. Factors for higher mode effects as well as coefficients for acceleration and velocity based foundation effects were introduced. A new set

of force modification factors for over-strength and ductility were included with a refined classification of seismic force resisting systems. Dynamic analysis was made the preferred method of analysis with restrictions for the equivalent static force procedure. The 2010 NBCC and 2015 NBCC (NRCC 2015) requirements remained essentially the same with improved hazard values and some minor modifications. Figure 1 illustrates the variation of seismic base shear over the years for six reinforced concrete frame buildings, three in Vancouver, and the other three in Ottawa. The buildings have 2, 5 and 10 storey heights. Because there was no distinction in seismic zones between Ottawa and Vancouver prior to 1970, and there was no ductile design and detailing requirements prior to 1975, the design base shear in both cities were the same, and the buildings were designed to have conventional, non-ductile frame systems. The 1970 NBCC was the exception, which made a distinction between non-ductile and ductile structural systems though ductile design and detailing were not implemented. Figure 1 also includes design base shears after 1975 with two levels of ductility; fully ductile for Vancouver and moderately ductile for Ottawa. Further details of the evolution of NBCC seismic provisions are provided by Mitchell et al. (2010).

In addition to the evolution of seismic base shear calculations outlined in the NBCC, the requirements for design and detailing of reinforcement in concrete buildings, specified in CSA A23.3 (CSA), also evolved over the years. These requirements were first introduced in CSA A23.3-1973 (CSA 1973), which was referenced in the 1975 NBCC. Though ductile design and detailing requirements were first introduced in 1973, the ductility based K factor to account for different types of construction had been introduced in the 1970 NBCC. Hence, buildings constructed prior to 1975 were considered in the current investigation as old conventional buildings. Significant improvements were introduced in CSA A23.3 (CSA 1984) after 1984 with

the introduction of capacity design requirements, protecting critical elements and preventing non-ductile failures.

Considering the evolution of seismic design practices in Canada, summarized above, the concrete frame building inventory may be viewed in two categories; buildings designed prior to 1975, and those designed using the ductile design requirements of the post-1975 era. The current paper focuses on buildings in the former category. It includes seismic assessment of non-ductile frame buildings built between 1953 and 1975. The design base shear remained approximately the same in this period, with lowest values given by the 1965 NBCC. The 1965 design was taken as the representative design for the era. This would result in somewhat conservative assessment of buildings designed for higher base shears. Table 1 includes the details for column and beam designs based on the 1965 NBCC.

The non-ductile frame buildings designed for the 1965 NBCC were modelled for non-linear dynamic analysis. The structural models were then used to conduct non-linear time history analysis using computer software PERFORM-3D (CSI 2013). The analysis was repeated under different earthquake intensities to implement the incremental dynamic analysis procedure from which fragility curves were obtained through probabilistic evaluation of results.

The fragility curves derived are compared with those developed earlier for the 2010 NBCC buildings (Al Mamun and Saatcioglu 2017) to illustrate differences in seismic vulnerabilities of older and newer frame buildings in Canada. In addition, two other comparisons are made to assess the significance of the change in building design practices between 1965 and 2010. First, the mean annual frequency of collapse was computed for each set of structures, and compared. Secondly, push-over analyses were performed to assess the performance under incrementally increasing static forces. The results are presented and discussed in the paper.

2. Building Details and Analysis Approach

2-, 5- and 10-storey reinforced concrete frame buildings with 8 m, 20 m and 40 m heights, respectively, were considered as representative structures for buildings having regular structural layouts. All the buildings had identical 5 bays in each direction; having a 7 m span, resulting in a 35 m by 35 m square plan. Figure 2(a) shows a three dimensional view of the 5-storey building modeled in ETABS software (CSI 2008). The buildings were designed for 1.33 kPa superimposed dead load in addition to their self-weight, and 2.44 kPa live load. The lateral seismic base shear (V) was calculated according to the equation provided in the 1965 NBCC, $V = RCIFSW$, where seismic regionalization factor $R = 4$, construction factor $C = 0.75$, importance factor $I = 1$, foundation factor $F = 1$, structural flexibility factor $S = 0.25/(N+5)$ for N -storey structure and weight of the structure W . Which resulted in the same design for buildings in Vancouver and Ottawa, as there was no distinction between the two regions at the time. The buildings were analyzed for the load combinations specified in the code. The compressive strength of concrete (f'_c) used was 30 MPa for 2- and 5-storey buildings and 40 MPa for the 10-storey building. Yield strength of reinforcement was 400 MPa for all designs.

Seismic records selected for non-linear dynamic analysis were compatible with the UHS specified in the 2010 NBCC. They consisted of 20 earthquake records (Gail Atkinson 2009) for each of the two regions considered, i.e., eastern and western Canada, adapted to the UHS for Ottawa and Vancouver, respectively. 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 15-25 km and 50-100 km. 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 15-25 km and 50-100 km. The algorithm proposed by Vamvatsikos and Cornell (2004) was used to

increase or decrease the intensity of records to perform incremental dynamic analysis (IDA). Further details of the seismic records and the implementation of the IDA are provided by the authors relative to the fragility analysis of newer buildings that conform to the 2010 NBCC (Al Mamun and Saatcioglu 2017).

The fragility analysis required a seismic intensity measure and damage indices for performance limits. The intensity measure used in the IDA was the spectral acceleration at the fundamental period (T_e) of the structure. The 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 (IO), ii) Life Safety (LS), and iii) Collapse prevention (CP) (ASCE 2014; FEMA 2000a; ACI 2013). Immediate Occupancy (IO) 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 drift of 1% was considered for this limit state. Life Safety (LS) performance level describes the damage state where significant damage has occurred to the structure with extensive cracking and hinge formation in primary structural elements. The inter-storey drift of 2% was adopted for this limit state. Collapse Prevention (CP) 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. This approach was also used by Ellingwood et al. (2007). The maximum inter-storey drift for CP limit was obtained as the smaller of the drift at which either dynamic instability is attained or the tangential slope of the IDA curve dropped to 20% of the initial slope (FEMA 350).

The conditional probability of exceeding a certain limit state (LS) at a given spectral acceleration ($S_a(T_e)$) was expressed by Jeong et al. (2007) as shown below in Equation 1.

$$(1) \quad P(D/IM) = 1 - \Phi\left(\frac{\ln D_C - \ln D_M}{\sqrt{\left(\frac{\beta_D^2}{M} + \beta_C^2 + \beta_M^2\right)}}$$

Where, D_C is the median drift capacity specified for a certain limit state. $\beta_{D/M}$ denotes standard deviation of log of deformation demand and assumed to be constant over the spectral acceleration range; β_C denotes standard deviation of log of deformation capacity and is taken as 0.3 (Jeong and Elnashai 2007) for all limit states, and β_M denotes uncertainty in analytical modeling and is taken as 0.2 on the assumption that the modelling process yields frame response with 90% confidence and within 30% of actual value (Ellingwood et al. 2007); D_M is the median drift demand and is defined as $D_M = a.S_a(T_e)^b$ where a and b are regression coefficients that are found from the ‘cloud’ analysis.

The outcome of fragility analysis was used to establish the mean annual frequency of collapse (λ_c). λ_c was calculated for each structure by integrating collapse fragility curve with site specific seismic hazard. The same approach was used by previous researchers to assess seismic collapse performance of buildings (Bradley and Dhakal 2008; Goulet et al. 2007; Haselton 2006). The following equation specified by Ibarra and Krawinkler (2005) was used to calculate λ_c :

$$(2) \quad \lambda_c = \int_0^\infty F_{C,S_{a,C}}(x) \cdot \nu \cdot f_{S_a}(x) \cdot dx$$

where $F_{C,S_{a,C}}(x)$ represents probability of exceeding certain spectral acceleration, x , corresponding to the collapse fragility curve, ν represented annual probability of exceedance of certain rate of seismicity, x and $f_{S_a}(x)$ represented the probability density function at certain spectral acceleration value, x . Return periods associated with 2% and 10% probability in 50-year

hazard values for spectral accelerations for Ottawa and Vancouver were obtained from Geological Survey of Canada (Adams and Halchuk 2003), and were used to calculate annual probability of exceedance (v).

Pushover analysis was also performed to determine the static lateral seismic force resistances of structures. Yield base shear capacity was established through idealized bilinear force deformation relationship according to FEMA 356 (FEMA 2000). Since the cracked stiffness was used to model structural members, linear cracked slope of the load deformation curve was found as effective stiffness of the structure. Yield shear force (V_y) and yield roof drift (δ_y) in idealized bilinear curve was calculated by using site specific UHS in the 2010 NBCC. From push-over analysis, over-strength factor (R_o) for each building was calculated as the ratio of yield shear capacity (V_y) to design base shear (V_d). Displacement ductility (μ) of a building was calculated as the ratio of maximum roof drift (δ_u) corresponding to 0.8 times the maximum shear strength (V_{max}) to yield roof drift (δ_y).

3. Analytical Models

The buildings were modeled using software PERFORM-3D. Beams and columns were modeled using two segments, each consisting of an elastic frame element with an inelastic hinge at the end, connected together at the point of inflection. Figure 2(b) shows the schematic diagram for a reinforced concrete frame in the analytical model. Elastic frame elements were assigned linear elastic properties with plastic deformations lumped at the hinges. Both flexural and shear hinges were provided. A rigid end zone was specified at the end of each structural element to eliminate deformations within the adjoining member, ensuring that potential plastic hinges form at the face of the adjoining member rather than at their centrelines. Energy degradation factor

(EDF) was used to introduce stiffness degrading hysteretic behaviour in flexure and shear as typically observed in reinforced concrete elements.

A tri-linear envelop curve was used for modelling moment-rotation hysteretic relationship. The initial linear segment represented the effective cracked stiffness based on 35% and 70% of uncracked flexural rigidities for beams and columns, respectively as per CSA A23.3-14. The reduction in stiffness values was suggested in the NBCC-1965, but no specific value was indicated for the amount of reduction to be applied. The yield moment was calculated from sectional analysis with the corresponding chord rotation at yield calculated as $\frac{ML}{6EI}$, where M represents the yield moment, L represents the member length, E represents the elastic modulus of concrete, and I represents the effective moment of inertia of the section. The second linear segment in the post yield range was established by assuming post-yield slopes of 8.5% and 4% of effective elastic slopes for beams and columns, respectively. The post yield segment continued up to an ultimate rotation of 1.5 times the yield rotation prior to the onset of strength decay. The descending branch, which ended at the residual strength, was based on the ASCE41-13 recommendation. The strength decay slope for both beams and columns was adopted from ASCE41-13 (ASCE 2014) for flexure-controlled response with nonconforming transverse reinforcement. The residual strength for beams was specified as 20% of maximum capacity. It varied between 0% and 20% of maximum capacity for columns depending on the accompanying level of axial load. Figure 3 illustrates the moment-rotation envelope used for the model in PERFORM-3D.

Once the primary curve was defined, the next step involved the computation of energy degradation factor (EDF), which introduced stiffness degradation characteristics of reinforced-concrete elements under reversed cyclic loading. EDF is the ratio of area under degraded cycle to

fully elastoplastic non-degraded cycle. This factor changes the hysteretic behaviour from perfectly elasto-plastic behaviour to stiffness degrading behaviour that is typically observed in reinforced concrete elements as illustrated in Fig. 4. The value for EDF was established by examining experimental data. Column tests conducted by Ozcebec and Saatcioglu (1989) were used for this purpose. The force-displacement hysteretic response of test column U1, obtained by testing a 350 mm square column having 1.0 m shear span, 3.3% longitudinal reinforcement and 0.85% transverse reinforcement, was used to define the flexural EDF as shown in Fig. 4(a). Accordingly the EDF value of 0.50 was found to match the experimental data. The same EDF value was used for both beams and columns.

The shear behaviour was also modelled with a tri-linear envelop curve for both beams and columns. A linear variation was considered for shear force-shear distortion relationship up to concrete shear capacity (V_c) and corresponding shear strain (γ_c), where $\gamma_c = \frac{V_c}{GA_v}$, A_v is shear area and G is shear rigidity. The second linear segment continued with a positive slope, but with a reduced shear rigidity $G_E = 0.1G$. This branch continued up to total shear capacity ($V = V_c + V_s$), consisting of concrete and transverse steel contributions (V_s). The total shear strain (γ) at this level was calculated as $\gamma = \gamma_c + \gamma_s$ where $\gamma_s = \frac{V_s}{G_E A_v}$. Shear displacement (Δ) was calculated as $\Delta = 0.5\gamma h$ where $0.5h$ corresponds to hinge length (50% of member depth, h). The third line segment with a descending slope was obtained from the ASCE41-13 recommendation specified for beam-column joints. The values specified in ASCE 41-13 for elements with nonconforming transverse reinforcement and axial loads of less than 10% of $A_g f'_c$ (where A_g is gross concrete area) were used for both beams and exterior columns. For the interior columns, the value for axial loads greater than 40% $A_g f'_c$ was used. Residual shear capacity was considered to be 20%

of beam and exterior column shear capacities, and 0% for interior column capacity. It was found that the degradation slope of shear force-shear displacement backbone curve was between 2 to 3% of initial slope for both beams and columns. The EDF for shear was based on experimental data. The hysteretic shear force-displacement relationship experimentally obtained by Ozcebec and Saatcioglu (1987) was used as shown in Fig. 4(b). Among the columns tested, the test column U1 was considered as representative of shear behaviour. The same EDF values were used both for beams and columns with values of 0.44 up to the end of the first line segment and 0.29 thereafter.

4. Performance Assessment of the Buildings

The results of dynamic analysis of buildings designed based on the 1965 NBCC indicated that beam hinging was observed as early as 1% storey drift, with the first storey columns also experiencing inelasticity. This implies that at the IO performance level, inelastic hinging has already started. These buildings never developed the 2% inter-storey drift associated with LS performance; and failed prior to reaching this limit state. Indeed, the CP limit state was reached on average at 1.5% storey drift, indicating that older buildings could collapse prior to developing the assumed life safety performance. In the 2-storey building analysed, flexural hinges formed in the 2nd floor beams, which were followed by the 1st floor columns. A similar trend was observed in reaching ultimate moment capacities of members. For all cases, the beams developed failure at the 1st floor level first, followed by the 1st floor columns. Shear hinges formed in the 1st floor columns at the same time as flexural hinges. Though column shear demands exceeded concrete shear capacity, no shear failure was observed. The 5-storey building experienced shear hinging first in the 1st floor interior columns, followed by flexural hinging of the 2nd floor exterior beams. Subsequently, flexural hinging extended to the 2nd, 3rd and 4th floor beams, followed by the

flexural hinging of the 1st floor columns. The failure was caused by insufficient flexural capacity in the 1st floor columns. A similar trend was observed in the 10-storey building; flexural hinging started forming in the 2nd floor beams and then gradually propagated up to the 6th floor level. Shear hinging occurred in the ground floor columns, followed by flexural hinging. The failure was due to insufficient shear capacity of these columns under high intensity excitations. The beams were not critical in shear, and there was no beam-shear hinging observed in any of the buildings.

The fragility analysis indicates that the 1965 buildings in Vancouver show 93% to 99% probability of exceeding the IO performance level at 2010 NBCC hazard levels. The same buildings under the same intensity levels show 68% to 91% probability of exceeding the CP level. The 1965 buildings in Ottawa showed somewhat lower probabilities of exceedances, showing 11% to 30% probability of exceeding the IO limit, and 1% to 9% probability of exceeding the CP limit state. Figure 5 shows the fragility curves for all the 1965 buildings located in Vancouver and Ottawa. The same figure also includes the fragility curves developed by the authors (Al Mamun and Saatcioglu 2017) for the same buildings designed using the 2010 NBCC requirements with design details illustrated in Table 1. A similar comparison is given in Table 2. It can be observed from Table 2 that at IO performance level, all the 1965 buildings exhibit similar response as those for the 2010 buildings. At the CP limit state, the 1965 buildings in Ottawa and Vancouver show on the average 6% and 76% higher probabilities of exceedance relative to the 2010 buildings, respectively. Demand standard deviation ($\beta_{D/M}$) associated to develop fragility function for 1965 buildings was 0.19; and for 2010 buildings located in Ottawa and Vancouver was 0.20 and 0.19, respectively.

Further comparisons of older and more recent building performances indicate that the maximum inter-storey drift ratio ($\delta_{\max}$) attained in the 1965 buildings are lower than those for the 2010 buildings. This is shown in Table 2, and is expected because of the brittle performance of older buildings. It is noteworthy that all the buildings had approximately the same yield drift capacities. This implies that the ductility levels considered in design are reflected in the analysis results. The ultimate flexural rotations of structural elements in the 1965 buildings developed 1.5 times their yield rotations; whereas the members in the 2010 buildings showed 2.5 and 4.0 times the yield rotation for buildings in Ottawa and Vancouver corresponding to moderately and fully ductile buildings.

The comparisons extended to include annual probability of exceedances. Figure 6 shows the graphical representation for calculating annual probability of exceedance (v) for the 2-storey 2010 building in Vancouver. Figure 7 graphically shows the approach for calculating the mean annual frequency of collapse (λ_c) for the same building at design spectral acceleration corresponding to the empirical code period ($S_a(T_d) = x$). In the figure, $f_{S_a}(x)$ is developed from probability distribution function at $S_a(T_d)$ observed in IDA and $F_{C,S_a,c}(x)$ is obtained from fragility response. Table 2 indicates that the 2010 Ottawa buildings have the lowest mean annual frequency of collapse (λ_c), whereas the 1965 Vancouver buildings have the highest λ_c .

Push-over analysis was also used to compare the performances of the 1965 and 2010 buildings. Figure 8 shows the comparison of push-over responses for 2-, 5- and 10-storey buildings. Figure 9 shows a bilinear idealization of the push-over response for the 2-storey 2010 building in Vancouver. Table 2 compares shear force responses obtained through push-over analyses of buildings. It indicates that the 2010 buildings in Vancouver have higher shear capacities, relative to the 1965 buildings. On average, the maximum shear capacity of 2010

Ottawa and Vancouver buildings are 26% and 56% higher than those for the 1965 buildings, respectively. Similar trend is observed for the maximum roof drift ratio (δ_u). Ottawa buildings show lower maximum roof displacements relative to the buildings in Vancouver. Over-strength ratio (R_o) obtained from the push-over analysis of 2010 Ottawa and Vancouver buildings are on average 39% and 76% higher than those for 1965 buildings, respectively. For the 5- and 10-storey 1965 buildings, the maximum shear capacity (V_{max}) is less than the design shear strength (V_d), which implies over-strength ratios (R_o) of less than 1.0. Insufficient shear capacity and low ductility of structural elements explain the poor performance of 1965 buildings observed in the comparison. Though structural members were detailed for a ductility-related force modification factors of 1.5 for the 1965 building, and 2.5 and 4.0 for the 2010 buildings in Ottawa and Vancouver, respectively, the average global ductility demands based on roof displacements are 1.6 for the 1965 buildings, and 2.7 and 2.6 for the 2010 buildings in Ottawa and Vancouver, respectively.

Figures 10(a) and 11(a) show the effect of global ductility ratio (μ) on mean annual collapse frequency (λ_c) of buildings. It is observed that for a given building; the mean annual collapse frequency decreases with increasing building ductility. Similarly, the mean annual collapse frequency (λ_c) decreases with increasing over-strength ratio (R_o), as shown in Figs. 10(b) and 11(b). The same trend was observed for all building height considered (2-, 5- and 10-storey). This suggests that the seismic safety margin can be increased with increasing ductility (μ) and over-strength (R_o).

The fragility analysis included the uncertainties involved in developing analytical models in predicting deformation demands and capacities of buildings. However, the fragility response might be adjusted due to the variations in material strength in older buildings. Rajeev and

Tesfamariam (2012) indicated that for 3-storey to 9-storey buildings with a wide variation in the properties of construction materials used (40% reduction in concrete strength and steel strength as well as 40% change in ties and stirrup spacing) resulted 29% change in deformation demands.

One of the limitations of the analytical model employed in developing the fragility curves was the assumption of rigid beam-column joints. Joint behaviour in pre-seismic design era can be affected by excessive cracking of joints, depending on numerous design parameters and the resulting softening in joints. Bonacci and Pantazopouou (1993) investigated the significance of joint softening in reinforced concrete frame buildings by examining a wide range of parameters. They concluded that it was difficult to draw a generalized conclusion on the effect of joint softening on structural response while pointing out that a large number of parameters affect joint behavior. The researchers also indicated that the effect of joint softening can be small for frames with weak beams, where the energy dissipation is concentrated in the beam hinge regions, and yet can be significant if extensive diagonal joint cracking occurs in the absence of joint transverse reinforcement, especially if the column size is small with reduced concrete shear resistance in joints. Clearly, the softening effect of beam-column joints increases with increased inelastic drift demands.

5. Summary and Conclusions

Three reinforced concrete frame buildings were designed based on the 1965 NBCC for seismic vulnerability analysis of buildings in Ottawa and Vancouver, as representative cities located in medium and high seismic regions in Canada. The buildings had 2-, 5- and 10-storey heights, and were designed as conventional buildings without the ductile seismic detailing required in current seismic codes. Dynamic inelastic response history analyses were conducted

using computer software PERFORM-3D under site specific earthquake records. Incremental dynamic analyses were performed under incrementally increasing seismic intensity, based of spectrum intensity as the intensity measure. The results were evaluated for different performance levels as dictated by corresponding damage indices, where inter-storey drift was adopted as the damage indicator. Commonly accepted performance levels, defined as immediate occupancy, life safety and collapse prevention were used. It was observed that the collapse prevention limit state was exceeded prior to developing the commonly accepted 2% storey drift as the life safety limit. Probabilistic analysis of the results of incremental dynamic analysis provided data to generate fragility curves that can be used as analytical tools for seismic vulnerability assessment. Fragility curves were developed and presented for six 1965 buildings, three in each location, for immediate occupancy and collapse prevention as performance levels. The fragility curves are compared with those generated in an earlier phase of the same investigation by the authors. The results indicate that the frame buildings in Vancouver showed very high probability of collapse when investigated under the 2010 NBCC seismic hazard prior to reaching the life safety limit of 2% of inter-storey drift. The frame buildings in Ottawa showed very small probabilities (on average 6%) of exceeding the collapse prevention limit under the same level of seismic hazard. All the buildings experienced inelastic shear deformations in their first storey columns with the 10-storey building experiencing shear failure. The beams remained elastic in shear but developed flexural hinges. The buildings showed poor performance relative to those designed and analysed earlier on the basis of the ductile design requirements of the 2010 NBCC. The comparisons and the specific probabilities of exceedances for different levels of performance are indicated in the paper. As indicated earlier, the fragility curves were developed for rigidly connected frame elements that do not experience significant softening in beam-column joints. The effect of

potential softening in joints with possible elongation in building period and its effect on spectral acceleration as seismic intensity measure can be accounted for approximately by the user through selecting a longer building period. Other parameters that pertain to potential variability in building materials and construction practices are not included in the fragility analysis and the buildings were assumed to fully conform to the requirements of their design NBCC and the prevailing construction practices of the era.

In addition to the fragility based assessment, two other approaches were employed for seismic performance assessment of 1965 buildings; annual frequency of collapse and push-over analysis. Both approaches showed agreement with the fragility analysis, indicating similar trends, i.e., significantly poor performance of 1965 buildings relative to 2010 buildings, both for western Canada (as represented by Vancouver buildings) and eastern Canada (as represented by Ottawa Buildings). It may be concluded that the fragility curves generated provide useful analytical tools for pre-1975 reinforced concrete frame buildings in Canada having regular structural layouts within a wide range of fundamental periods.

References

- ACI. 2013. Guide for testing reinforced concrete structural elements under slowly applied simulated seismic loads. ACI committee 374. American Concrete Institute. ACI 374.2R-13

- Al Mamun, A. and Saatcioglu, M. 2017. Seismic fragility curves for reinforced concrete frame buildings in Canada designed after 1985. *Canadian Journal of Civil Engineering*. Volume 44, Number 7, page 558-568.
- ASCE. 2014. Seismic evaluation and retrofit of existing buildings. American Society of Civil Engineers. ASCE/SEI 41-13.
- Adams, J. and Halchuk, S. 2003. Fourth generation seismic hazard maps of Canada: Values for over 650 Canadian localities intended for the 2005 National Building Code of Canada. Geological Survey of Canada. Open file 4459.
- Atkinson, G.M. 2009. Earthquake time histories compatible with the 2005 National building code of Canada uniform hard spectrum. *Canadian Journal of Civil Engineering*. Volume 36, Number 6, June 2009.
- Bonacci, J. and Pantazopoulou, S. Parametric investigation of joint mechanics. *ACI Structural Journal*, V. 90, N0.1, 1993, Pages 61-71.
- Bradley, B. and Dhakal, R. 2008. Error estimation of closed-form solution for annual rate of structural collapse. *Earthquake Engineering & Structural Dynamics*. Volume 37, Issue 15. Pages 1721-1737.
- Bilgin, Huseyin. 2013. Fragility-based assessment of public buildings in Turkey. *Engineering Structures* no. 56:1283-1294.
- Buratti, N., Ferracuti, B. and Savoia, M. 2010. Response surface with random factors for seismic fragility of reinforced concrete frames. *Structural Safety*. Volume 32, Issue 1, Pages 42-51.
- Huseyin Bilgin. 2013. Fragility-based assessment of public buildings in Turkey. *Engineering Structures*. Volume 56, Pages 1283-1294.
- CSA. 1973. Design of concrete structures. CSA standards update service. Canadian Standards Association. CSA A23.3-1973. Mississauga, ON.
- CSA. 1984. Design of concrete structures. CSA standards update service. Canadian Standards Association. CSA A23.3-1984. Mississauga, ON.
- CSI. 2008. ETABS. Nonlinear Version 9.5.0. Computers and Structures, Inc. Berkeley, CA.
- CSI. 2013. PERFORM-3D. Version 5.0.1. Computers and Structures, Inc. Berkeley, CA.
- Ellingwood, B.R., Celik, O.C., and Kinali, K. 2007. Fragility assessment of building structural systems in Mid-America. *Earthquake Engineering and Structural Dynamics*. **36**(13):1935-1952. doi: 10.1002/eqe.693
- FEMA. 2000a. Prestandard and commentary for the seismic rehabilitation of buildings. Federal Emergency Management Agency. FEMA 356. Washington, D.C.
- FEMA. 2000b. Recommended seismic design criteria for new steel moment-frame buildings. SAC Joint Venture. Federal Emergency Management Agency. FEMA 350. Washington, D.C.

- 453 Goulet, C.A., Haselton, C.B., Reiser, J.M., Beck, J.L., Deierlein, G.G., Porter, K.A. and Stewart, J.P. 2007. Evaluation
454 of the seismic performance of a code-conforming reinforced-concrete frame building—from seismic hazard to
455 collapse safety and economic losses. *Earthquake Engineering & Structural Dynamics*. Volume 36, Issue 23.
456 Pages 1973-1997.
- 457 Haselton, Curt B. 2006. Assessing seismic collapse safety of modern reinforced concrete moment frame buildings.
458 Stanford University. Ph.D. Thesis.
- 459 Ibarra, L.F. and Krawinkler, H. 2005. Global collapse of frame structures under seismic excitations. Department of
460 Civil and Environmental Engineering. Stanford University. Final report on PEER project 3192002. Report no.
461 152.
- 462 Jeong, S. H., and Elnashai, A.S. 2007. Fragility relationships for torsionally-imbalanced buildings using three-
463 dimensional damage characterization. *Engineering structures*. Volume 29, Issue 9, Pages 2172-2182.
- 464 Lupoi, G., Franchin, P., Lupoi, P and Pinto, P. 2006. Seismic fragility analysis of structural systems. *Engineering*
465 *Mechanics*. Vol 132, Issue 4, Page 385-395.
- 466 Mitchell, D., Paultre, P., Tinawi, R., Saatcioglu, M., Tremblay, R., Elwood, K., Adams, J., and DeVall, R. 2010.
467 Evolution of seismic design provisions in the national building code of Canada. *Canadian Journal of Civil*
468 *Engineering*. Volume 37, Issue 9, Pages 1157-1170
- 469 NRCC. 1941. National building code of Canada 1941. Associate Committee on the National Building Code.
470 National Research Council of Canada, Ottawa, ON.
- 471 NRCC. 1953. National building code of Canada 1953. Associate Committee on the National Building Code.
472 National Research Council of Canada, Ottawa, ON.
- 473 NRCC. 1965. National building code of Canada 1965. Associate Committee on the National Building Code.
474 National Research Council of Canada, Ottawa, ON.
- 475 NRCC. 1970. National building code of Canada 1970. Associate Committee on the National Building Code.
476 National Research Council of Canada, Ottawa, ON.
- 477 NRCC. 1975. National building code of Canada 1975. Associate Committee on the National Building Code.
478 National Research Council of Canada, Ottawa, ON.
- 479 NRCC. 1977. National building code of Canada 1977. Associate Committee on the National Building Code.
480 National Research Council of Canada, Ottawa, ON.
- 481 NRCC. 1980. National building code of Canada 1980. Associate Committee on the National Building Code.
482 National Research Council of Canada, Ottawa, ON.
- 483 NRCC. 1985. National building code of Canada 1985. Associate Committee on the National Building Code.
484 National Research Council of Canada, Ottawa, ON.

- 485 NRCC. 1990. National building code of Canada 1990. Associate Committee on the National Building Code.
486 National Research Council of Canada, Ottawa, ON.
- 487 NRCC. 1995. National building code of Canada 1995. Associate Committee on the National Building Code.
488 National Research Council of Canada, Ottawa, ON.
- 489 NRCC. 2005. National building code of Canada 2005. Associate Committee on the National Building Code.
490 National Research Council of Canada, Ottawa, ON.
- 491 NRCC. 2010. National building code of Canada 2010. Associate Committee on the National Building Code.
492 National Research Council of Canada, Ottawa, ON.
- 493 NRCC. 2015. National building code of Canada 2015. Associate Committee on the National Building Code.
494 National Research Council of Canada, Ottawa, ON.
- 495 Ozcebe, G., Saatcioglu, M. 1987. Confinement of concrete columns for seismic loading. ACI Structural Journal.
496 Volume 84, Issue 4, Pages 308-315.
- 497 Ozcebe, G., Saatcioglu, M. 1989. Hysteretic shear model for reinforced concrete members. Journal of the Structural
498 Engineering, ASCE. Volume 115, Issue 1, pp. 132-148.
- 499 Rajeev, P., and Tesfamariam, S. 2012. Seismic fragilities for reinforced concrete buildings with consideration of
500 irregularities. Structural Safety. <https://doi.org/10.1016/j.strusafe.2012.06.001>
- 501 Ramamoorthy, S. K., Gardoni, P. and Bracci, J. M. 2008. Seismic fragility and confidence bounds for gravity load
502 designed reinforced concrete frames of varying height. Journal of Structural Engineering. Volume 134, Issue 4,
503 Pages 639-650.
- 504 Vamvatsikos, D., and Cornell, C.A. 2004. Applied incremental dynamic analysis. Earthquake Spectra. Vol 20, No.
505 2, Page 523-553, May 2004.
- 506 Ventura, C., Finn, L., Onur, T., Blanquera, A., and Rezai, M. 2005. Regional seismic risk in British Columbia-
507 classification of buildings and development of damage probability functions. Canadian Journal of Civil
508 Engineering. Volume 32, Issue 2, pages 372-387.

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Table 1. Design details for columns and beams of 1965 and 2010 buildings

	2 Storey (40MPa)		2 Storey (30MPa)		5 Storey (30MPa)		5 Storey (30MPa)		10 Storey (40MPa)			10 Storey (40MPa)	
	2010		1965		2010		1965			2010 (Ottawa)	2010 (Vanc.)	1965	
	Size	Rebar	Size	Rebar	Size	Rebar	Size	Rebar	Size	Rebar	Rebar	Size	Rebar
Corner Column 1-Top	300X300	8-20M	300X300	4-25M+4-20M	300X300	8-20M	300X300	4-25M+4-15M	350X350	4-25M+4-15M	4-25M+4-15M	350X350	8-20M
Ext Column 1-2	300X300	4-30M	300X300	8-25M	300X300	4-30M	325X325	8-25M	350X350	16-20M	4-30M+8-20M	400X400	8-25M
Ext Column 3-Top	-	-	-	-	300X300	4-25M	325X325	4-25M+4-20M	350X350	4-25M+4-15M	8-25M	400X400	4-25M+4-20M
Int-1 Column 1-2	325X325	4-20M+4-15M	300X300	8-15M	450X450	12-25M	400X400	8-30M	500X500	12-30M+8-20M	12-30M+8-25M	500X500	12-25M+4-20M
Int-1 Column 3-5	-	-	-	-	450X450	4-25M+4-20M	400X400	4-25M+4-15M	500X500	8-25M+8-20M	8-30M+8-25M	500X500	8-25M
Int-1 Column 6-10	-	-	-	-	-	-	-	-	500X500	4-25M+4-15M	4-25M+4-20M	500X500	4-25M+4-15M
Int-2 Column 1-2	325X325	4-20M+4-15M	300X300	8-15M	450X450	4-30M+8-20M	400X400	4-30M+4-25M	500X500	12-30M+8-20M	12-30M+8-20M	500X500	8-25M+8-20M
Int-2 Column 3-5	-	-	-	-	450X450	4-30M	400X400	8-20M	500X500	8-25M+8-15M	12-25M+8-20M	500X500	4-25M+8-15M
Int-2 Column 6-10	-	-	-	-	-	-	-	-	500X500	4-25M+4-15M	4-25M+4-20M	500X500	4-25M+4-15M
Int-3 Column 1-2	325X325	4-20M+4-15M	300X300	4-20M	450X450	4-25M+8-20M	400X400	8-25M	500X500	8-30M+8-25M	12-30M+8-20M	500X500	12-25M
Int-3 Column 3-5	-	-	-	-	450X450	8-20M	400X400	8-20M	500X500	4-25M+8-20M	12-25M+4-20M	500X500	4-25M+4-15M
Int-3 Column 6-10	-	-	-	-	-	-	-	-	500X500	4-25M+4-15M	4-25M+4-20M	500X500	4-25M+4-15M
Ext Beam Top	300X500	3-20M	300X500	4-20M	300X500	3-20M	300X500	4-20M	350X500	3-20M	3-20M	350X500	2-25M
Ext Beam Bottom	300X500	2-20M	300X500	2-20M	300X500	2-20M	300X500	2-20M	350X500	2-20M	2-20M	350X500	2-20M
		Ottawa	Vanc.										
Int Beam 1-5 Top	300X500	4-20M	4-20M	300X500	3-25M	300X500	3-25M	300X500	3-25M	350X500	4-25M	350X500	5-25M
Int Beam 1-5 Bottom	300X500	3-20M	4-20M	300X500	3-15M	300X500	2-25M	300X500	3-20M	350X500	2-25M	350X500	4-20M
Int Beam 6-10 Top	-	-	-	-	-	-	-	-	350X500	3-25M	3-25M	350X500	4-25M
Int Beam 6-10 Bottom	-	-	-	-	-	-	-	-	350X500	3-20M	3-25M	350X500	2-25M

Table 2. Comparisons of vulnerability assessment for 1965 and 2010 buildings

Design Code	Building	Region	T_e	T_d	$S_a(T_e)$	$S_a(T_d)$	δ_{max} (%)	Regression Coefficient			LS Prob. at $S_a(T_e)$		λ_c at $S_a(T_d)$ $\times 10^{-04}$	V_y Kn	δ_y (%)	V_d Kn	δ_u (%)	V_{max} Kn	μ	R_o
								a	b	$\beta_{D/M}$	IO	CP								
2010 NBCC	10-Storey	Ottawa	2.84	1.79	0.05	0.07	2.44	16.8	1.04	0.24	26	0	0	2438	0.26	1843	0.74	3468	2.8	1.3
	5-Storey	Ottawa	2.04	1.06	0.05	0.13	2.94	12.4	1.02	0.21	10	0	3	1983	0.48	1456	0.95	2738	2.0	1.4
	2-Storey	Ottawa	1.08	0.54	0.13	0.30	3.92	4.74	0.98	0.15	14	0	0	1667	0.54	1370	1.85	2413	3.4	1.2
	10-Storey	Vanc.	2.84	1.79	0.17	0.20	3.07	11.3	1.01	0.21	95	14	18	3790	0.37	2936	0.86	5123	2.3	1.3
	5-Storey	Vanc.	2.04	1.06	0.17	0.32	3.91	10.3	1.00	0.18	93	3	54	2424	0.59	1669	1.38	3031	2.4	1.5
	2-Storey	Vanc.	1.08	0.54	0.32	0.62	3.99	5.51	1.02	0.19	92	2	32	2296	0.78	1463	2.48	2509	3.2	1.6
1965 NBCC	10-Storey	Ottawa	2.81	1.79	0.05	0.07	1.53	20.2	1.08	0.46	30	8	30	2475	0.27	3438	0.35	2925	1.3	0.7
	5-Storey	Ottawa	2.15	1.06	0.05	0.13	1.04	12.9	0.99	0.40	11	9	160	1651	0.45	1770	0.60	1986	1.3	0.9
	2-Storey	Ottawa	1.14	0.54	0.13	0.30	1.80	5.48	0.99	0.38	18	1	82	1313	0.48	1097	1.15	1909	2.4	1.2
	10-Storey	Vanc.	2.81	1.79	0.17	0.20	1.69	23.1	1.16	0.48	99	88	121	2234	0.25	3438	0.35	2925	1.4	0.6
	5-Storey	Vanc.	2.15	1.06	0.17	0.32	1.27	13.2	1.00	0.41	97	91	163	1440	0.39	1770	0.60	1986	1.5	0.8
	2-Storey	Vanc.	1.14	0.54	0.31	0.62	1.48	5.26	0.98	0.38	93	68	222	1214	0.64	1097	1.15	1909	1.8	1.1

Note: T_d is calculated as $1.5 \times 0.075(h_n)^{3/4}$ where h_n is height of the structure.

δ_{max} is the maximum inter-storey drift corresponding to the collapse prevention (CP) limit state.

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(a)

(b)

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