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SEISMIC VULNERABILITY ASSESSMENT OF CANADIAN REINFORCED CONCRETE SHEAR WALL BUILDINGS DESIGNED IN PRE-MODERN SEISMIC CODE ERA

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Abstract: This research intends to assess seismic vulnerability of existing Canadian reinforced concrete shear wall buildings. A probabilistic methodology is taken to account to quantify the likelihood of exceeding different damage sates under different levels of seismic hazard intensity through seismic fragility curves. The buildings considered are designed prior to the enactment of ductile design requirements in Canada. Considering 1975 as the benchmark for the introduction of modern seismic design provision, pre-1975 shear wall buildings with regular plan, were simulated in PERFORM 3D software for nonlinear modelling and analysis. The buildings were subjected to Incremental Dynamic Analysis by applying time history records compatible with Canadian seismicity with increasing intensity to cover the whole range of structural behaviour. Seismic fragility curves of the existing shear wall buildings designed in pre-seismic code era were derived considering immediate occupancy, life safety and collapse prevention as damage states. It is concluded that Canadian shear wall buildings designed in pre-seismic code era are vulnerable based on life safety objective of recent modern seismic codes. Therefore, consideration of rehabilitation strategies based on acceptable margin of safety intended may be necessary.

1 Introduction

Incorporation of improved hazard values and ductile seismic design and detailing since 1975 resulted in significant progress in seismic design provisions in Canada. A large number of shear wall buildings exists in Canada, many designed and constructed prior to 1975. The current study provides seismic vulnerability assessment of older reinforced concrete shear wall buildings. Vulnerability assessment of post-1975 shear wall buildings are reported elsewhere (Rafie Nazari and Saatcioglu)[1]. Vancouver and Ottawa have been chosen as representative of western and eastern Canadian seismicity in the current investigation. Shear wall buildings having 2-, 5- and 10-storey heights were selected to cover a range of building periods used in practice.

[1] Rafie Nazari and Saatcioglu. 2016. Fragility curves for Canadian shear wall buildings conforming ductile requirements. Submitted for publication in *Canadian Journal of civil Engineering*.

There are few studies in literature on seismic risk assessment of buildings in Canada. Ventura et al. (2005) studied seismic risk estimation in southwestern British Columbia, to derive damage probability matrix and fragility curves for each building class in terms of Modified Mercali Intensity (MMI). Other research for seismic vulnerability assessment of reinforced concrete (RC) buildings elsewhere in the World mainly focused on RC frame buildings. Ramamoorthy et al. (2008) developed fragility curves for reinforced concrete frame buildings designed for gravity loads and estimated fragility as a function of spectral acceleration using peak inter-storey drift as the response parameter. Celik and Ellingwood (2009) assessed seismic vulnerability of gravity load designed RC frames in Memphis, Tenn. applying synthetic earthquake ground motions. They concluded that the majority of existing gravity load designed RC building in Memphis are vulnerable for life safety and collapse prevention performance objectives. Masi and Vona (Masi and Vona 2012) assessed seismic capacity of existing RC buildings designed for gravity loads. They applied non-linear dynamic analysis on selected structural models for buildings with different ages, number of storeys, either with or without infill walls. Bilgin (2013) studied seismic fragility of reinforced concrete public buildings with representative designs according to the 1975 version of the Turkish seismic design code considering peak ground velocity as seismic intensity measure. Jeon et al. (2015) studied fragility functions for non-ductile concrete frames using numerical simulation in Opensees and applying ground motions representative of the seismic hazard in California. Pitilakis et al. (2014) assessed seismic vulnerability of RC frame buildings considering the effects of building age and soil-structure interaction. Karapetrou et al. (2015) also studied the effect soil-structure interaction and site soil conditions on seismic performance of RC moment resisting frames.

2 Selection and Design of Representative Buildings

Three shear wall buildings with 2-, 5-, and 10-storey heights and regular plan were selected as representative buildings having pre-1975 Canadian code designs. The floor plan consisted of five bays with 7.0 m span length in each direction and storey height of 4.0 m. **Error! Reference source not found.** shows the geometric details of the buildings.

In order to assess seismic vulnerability of buildings designed within a time period, variation in seismic design force levels and the evolution of seismic design and detailing requirements need to be studied. In Canada, seismic design force levels are obtained from NBCC and design and detailing requirements are addressed in CSA A23.3. The design practice for shear wall buildings in Canada can be viewed in two categories: those performed prior to the enactment of ductile design principles (pre-1975 era), and those performed in the post 1975 era. The 1975 NBCC (NRCC 1975) made reference to CSA A23.3-1973 (CSA 1973), which was the first edition of the concrete design standard that implemented ductile design principles. The current paper provides seismic assessment of older existing buildings, designed between 1953 and 1975.

The selected shear wall buildings were designed conforming to the requirement of the 1965 NBCC (NRCC 1965) for Vancouver, and the 1970 NBCC (NRCC 1970) for Ottawa as the years giving minimum design load level between 1953 to 1970. All the buildings were designed to resist seismic lateral forces with two shear walls in each direction. The gravity loads included 1.9 kPa of live load and 1.0 kPa super-imposed dead load in addition to the weight of the structural elements. The designs were carried out for 25 MPa concrete, representing the common concrete strength of older structures, and grade 400 MPa reinforcing steel. These buildings were designed without confined wall boundary elements. They did not contain buckling prevention ties or sufficient transverse reinforcement for diagonal tension, and hence they could develop shear failure before flexural failure.

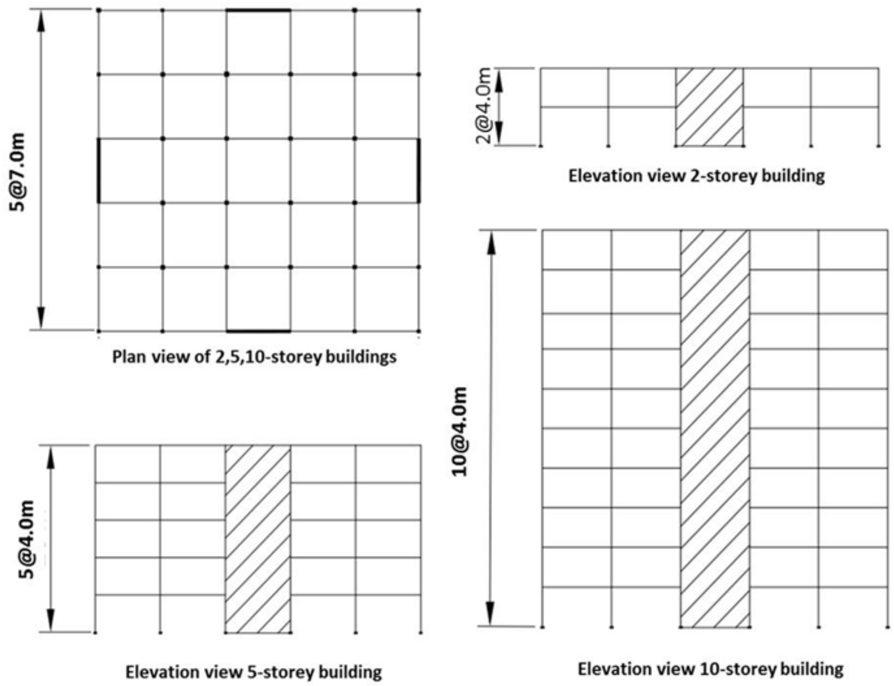

Figure 1: Plan and elevation views of selected buildings (Rafie Nazari and Saatcioglu)[2]

3 Nonlinear Modelling and Analysis

3.1 Nonlinear modelling of shear wall buildings

Computer software PERFORM 3D (CSI 2013), developed by Computers and Structures Inc., was used as the analytical tool for nonlinear dynamic analysis. Three dimensional numerical models were generated for the buildings using their design and detailing properties. PERFORM 3D has a variety of options for nonlinear modelling of elements, with inelasticity introduced either through plastic hinges or material non-linearity assigned to fiber models. FEMA Beam and FEMA Column components in PERFORM 3D allow the user to divide each structural component into a hinge and an elastic segment, satisfying the relationship between end moments and end rotations. Strength loss for beams and columns are enabled, which is as an essential characteristic for the current research, as the level of applied incremental load can go beyond the capacity for some of the elements. FEMA beam and FEMA column components were implemented to simulate nonlinear beam and column elements assuming lumped plasticity defined in plastic hinges. The initial effective rigidities of 0.35EI and 0.70EI were assigned to beams and columns, respectively. Nonlinear flexural characteristics of these members were defined using moment capacity obtained from the sectional analysis conducted using software SAP2000 (CSI 2009). Hinge properties were based on the ASCE 41 (ASCE 2007) recommendations for members without ductile detailing.

Response of shear wall buildings is primarily dominated by the behaviour of shear walls, which necessitates an accurate and detailed model for the walls. The shear wall model in PERFORM 3D was

[2] Rafie Nazari and Saatcioglu. 2016. Fragility curves for Canadian shear wall buildings conforming ductile requirements. Submitted for publication in *Canadian Journal of civil Engineering*.

verified earlier against tests of large-scale shear walls (Rafie Nazari and Saatcioglu)[3], and was found to be representative of the intended behavior. Accordingly, a fiber-discretized model was employed for the simulation of wall behaviour. This involved the integration of concrete and steel segments across the depth of the shear wall element. Hognestad's unconfined concrete (1951) model was used in the entire wall section as the pre-1975 walls did not have confined concrete. The analytical model also considered inelastic shear response. Perform 3D allows nonlinear shear behaviour in the form of stress-strain behaviour of shear wall section. This relationship was assigned to the wall models to improve the accuracy of computed displacements in highly non-linear range of deformations. The non-linear shear model had a trilinear shear material behaviour with a descending branch. The slope of the stress-strain relationship is assumed to be equal to the shear modulus "G" until the shear strength of concrete is attained. The second branch was modelled with a linear segment having a slope equal to 6% of the initial slope. This segment was then followed by a straight line as the descending branch.

3.2 Incremental Dynamic Analysis

Incremental dynamic analysis (Vamvatsikos and Cornell 2002) is employed to estimate structural performance under different intensities of earthquakes. These analyses involve subjecting the structural model to a set of ground motion records, each scaled to multiple levels of intensity, resulting in curves of response parameter versus intensity level.

Artificial earthquake ground motions, generated by Atkinson (2009) for western and eastern Canadian seismicity, were selected first to conduct incremental dynamic analysis (IDA). These records are compatible with the uniform hazard spectra (UHS) specified for seismic design in the 2005 and 2010 National Building Code of Canada. The records were generated for earthquakes having 2% probability of exceedance in 50 years. In this study, the records generated for the reference soil Type C were used. A detailed discussion of the record selection is available elsewhere (Rafie Nazari and Saatcioglu)[4]. A set of twenty records were selected for each site. Selected records were scaled to cover the entire range of structural response.

Incremental Dynamic analysis involve two core parameters: seismic intensity measure and engineering demand parameter (damage indicator). Spectral acceleration of fundamental period with 5% of critical damping ($S_a(T_1)$,5%) was selected as seismic intensity measure. $Sa(T_1)$ was used by previous researches including Vamvatsikos and Cornell (2002) and Ellingwood et al. (2007). This measure of intensity reflects both the characteristic of the earthquake and the structural period. It is defined in the National Building Code of Canada as a design parameter, and is frequently used by designers.

The analytical models with the inelastic member properties discussed above were used to conduct inelastic dynamic response history analysis under the ground motion records selected. The use of a set of twenty records with different scale factors resulted in over 200 dynamic analyses for each model. The IDA results are then used to develop seismic fragility curves. The generation of seismic fragility curves is presented in the following sections.

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[3] Rafie Nazari and Saatcioglu. 2017. Seismic performance assessment of shear wall buildings through fragility analysis. Submitted for publication in Engineering Structures.

[4] Rafie Nazari and Saatcioglu. 2016. Fragility curves for Canadian shear wall buildings conforming ductile requirements. Submitted for publication in *Canadian Journal of civil Engineering*.

4 Development of Fragility Curves

IDA results provide the input for developing fragility curves as probabilistic tools representing the probability of exceeding predefined damage states under different levels of ground motion intensity. Fragility function is described in the form of Eq. 1.

$$P[D > Di \mid IM] = \phi\left[\frac{\ln(x / Di)}{\sqrt{\beta_{d/IM}{}^{2} + \beta_{c}{}^{2} + \beta_{m}{}^{2}}}\right] \qquad \text{Eq. 1}$$

In the above expression, φ(.) is standard normal cumulative distribution function, D_i is upper bound for each damage level, x is the median value of demand as a function of IM, $\beta_{d/IM}$ is the dispersion (logarithmic standard deviation) of demand conditioned on IM, β_c is capacity uncertainty and β_m is modeling uncertainty. The fragility curves are implemented in the risk assessment methodology to make a judgement on the performance of structures. Each level of damage can be representative of a certain level of loss in terms of money and downtime.

IDA curves provide structural response. In order to assess of structural damage different levels of damage that the buildings experience need to be classified based on limit states. Herein, three different limit states are used; i) Immediate Occupancy, ii) Life Safety and iii) Collapse Prevention. The description of these damage states as per ASCE 41(ASCE 2007) is as follows:

•Immediate Occupancy: Building remains safe to be reoccupied. Lateral-force and gravity-load-resisting systems retain most of their design strengths.

•Life Safety: Building undergoes significant damage. Structural elements and components may be severely damaged, but gravity-load-carrying elements continue fulfilling their functions.

•Collapse Prevention: Substantial damage is imposed on structural elements. Significant strength and stiffness degradation of the lateral-load-resisting system is observed, and large permanent lateral deformations occur. The structure is not repairable and not safe to reoccupy.

These definitions give qualitative descriptions of damage states while quantitative definitions are required for the engineering demand parameters chosen. The first-storey drift has been selected as a damage parameter in the current research project. The ASCE 41 recommendation of 0.5% drift ratio for shear wall buildings was selected as the threshold for immediate occupancy. Observations made from the IDA curves confirm that until this drift value is exceeded the structural behaviour remains essentially linear, allowing the building to be immediately re-occupied. ASCE 41 suggests a value of 1.5% drift ratio for the collapse prevention limit state of shear wall buildings with possibility of shear failure which is employed in the current study. The threshold for life safety limit state is assumed to be 1% drift as recommended by ASCE 41 for shear wall buildings. This provides a good estimate for life safety performance as it is in the middle of the other two limit states.

Using the results of IDA and applying probabilistic analysis, fragility curves are derived for the selected buildings. Figure 3 show the results of fragility analyses for 2-, 5- and 10-storey shear wall buildings designed prior to 1975.

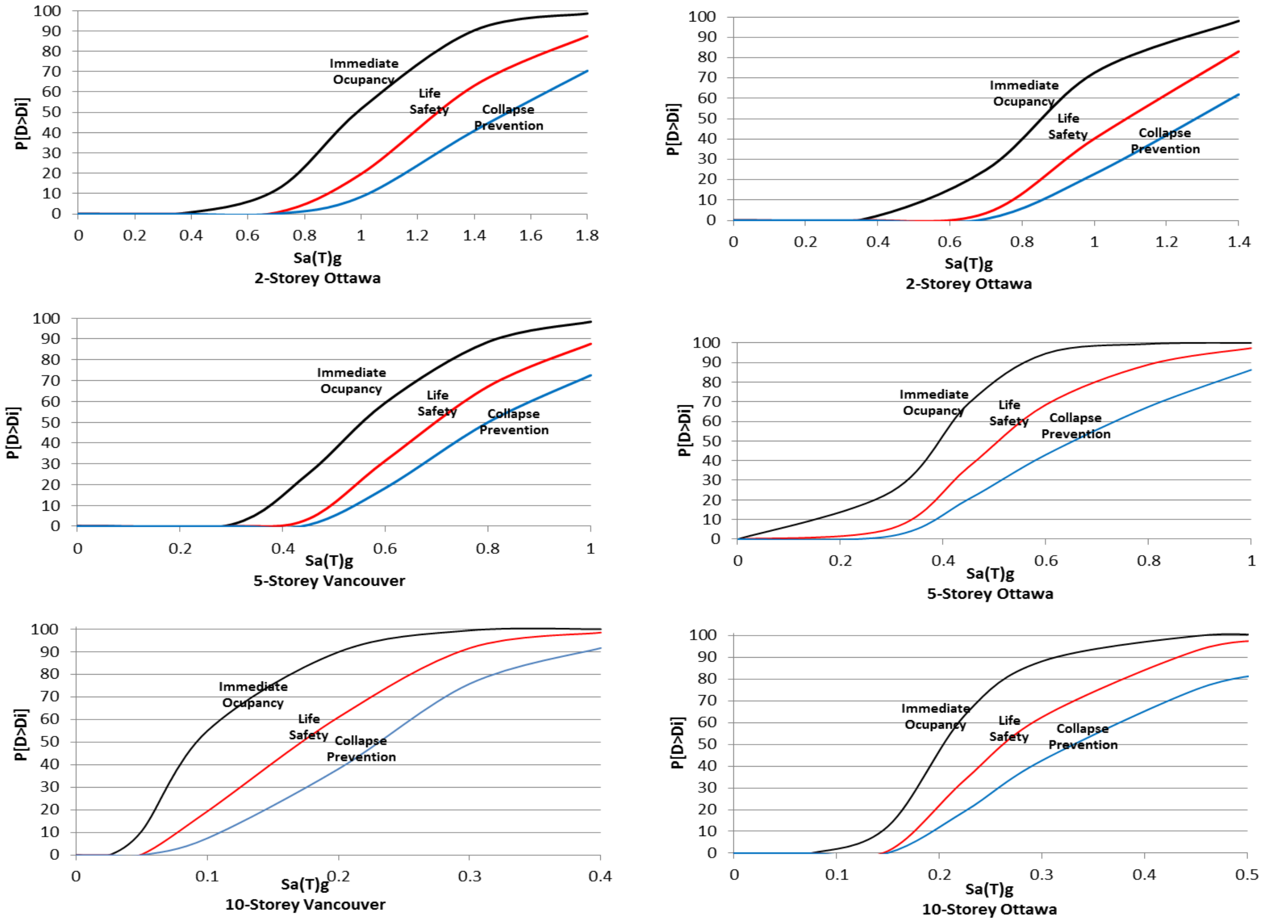

Figure 3: Fragility curves derived for buildings designed in pre-modern seismic code era

5 Discussion and Conclusion

A probabilistic methodology was implemented to generate analytical tools for seismic vulnerability assessment of shear wall buildings through fragility curves. Fragility curves quantify the probability of structural damage as a function of seismic hazard level. The curves were developed for reinforced concrete shear wall buildings designed in the pre-modern seismic code era in Vancouver and Ottawa. PERFORM3D software was used to generate nonlinear response quantities under two sets of twenty synthetic records for western and eastern Canadian seismicity. Incremental Dynamic Analysis procedure was employed to cover the entire range of structural response having spectral acceleration as the ground motion intensity measure and the first storey drift as the damage indicator. Table 1 summarizes probabilities of exceeding different damage limit states for the 2-, 5-, and 10- storey shear wall buildings considered. The probabilities of exceedances were assessed at spectral accelerations at structural periods based on 2% in 50 year hazard values (based on 2010 NBCC). The periods were computed by the empirical expressions given in the 2010 NBCC. The results indicate that shear wall buildings designed in pre-modern seismic code era in Canada show high vulnerabilities under the 2010 NBCC hazard values. Taller shear wall buildings in Vancouver show more vulnerability relative to mid-rise and low-rise buildings located in that city. Buildings in Vancouver are significantly more vulnerable than those located in Ottawa.

Table 1: Probability of exceedance for life safety at 2475 years hazard level

		Vancouver 1965	Ottawa 1970
Building	Limit State	Probability of exceeding	Probability of exceeding
2-Storey	IO	10%	5%
	LS	0%	0%
	CP	0%	0%
5-Storey	IO	35%	25%
	LS	10%	5%
	CP	5%	2%
10-Storey	IO	90%	10%
	LS	70%	0%
	CP	50%	0%

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