The 11th Canadian Conference on Earthquake Engineering
Canadian Association for Earthquake Engineering

# DEVELOPMENT OF SYSTEM-LEVEL COLLAPSE FRAGILITY CURVES FOR DUCTILE REINFORCED CONCRETE BLOCK STRUCTURAL WALLS SYSTEMS IN CANADA

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**ABSTRACT:** Within the framework of performance-based design of reinforced masonry structural walls (RMSW), seismic risk assessment is key to evaluating the vulnerability of RMSW systems under different levels of seismic demands. Current and previous experimental studies that focused on the performance of RMSW components have demonstrated their ductile response when designed and detailed according to prescriptive seismic code requirements. A recent component-level experimental study by the authors on RMSW quantified key seismic performance parameters of RMSW with different configurations. The RMSW were tested under fully reversed cyclic loading simulating seismic effects. The results show that even for RMSW detailed to meet the requirements of the same seismic design category, their seismic performance can vary significantly. To further examine system-level of RMSW, an archetype RMSW office building designed according to the NBCC 2010 and CSA S304-14 was utilized to develop collapse fragility curves. As a first step, using a distributed plasticity model within the OpenSees platform, an analytical model was developed and calibrated using experimental RMSW test results. Subsequently, incremental dynamic analysis (IDA) was performed to study the system-level RMSW performance under several synthetic ground motion records, with response spectra that match that of the NBCC 2010 design spectrum for Victoria, British Columbia. Using the IDA results, the median collapse intensity and dispersion were estimated and the system-level collapse fragility curves were fitted using the method of moment estimator.

## 1. Introduction

In the past few years there has been an evident shift towards probabilistic seismic risk assessment (PSRA) of masonry structures. This ideology is getting more prominent realizing the fact that nothing is certain when assessing the performance of a structure when subjected to an earthquake. Earthquakes are random events and even if it is possible to predict when a strong event will occur it is impossible to predict the intensity of such an event, how the structures will behave during such an event and the causalities, (social and economic) caused by the event. Due to the large amount of uncertainties involved in the process of performance assessment of buildings, the Federal Emergency Management Agency (FEMA) took further steps in documenting a methodology to quantify performance of buildings. Significance portion of the methodology will be used to assess performance of RM masonry structures. Since there is no existing Canadian performance tool at the moment, the FEMA procedure will be followed using Canadian design spectrum-consistent records that have been used to generate Seismic Hazard curves for the National Building Code of Canada (Assatourians and Atkinson 2010). Collapse fragility assessment, considered to be one of the steps of the performance measures was performed to quantify the vulnerability of such structures when subjected to strong ground motions. The focus of this study is to quantify collapse of low-rise reinforced masonry shear wall buildings by using the collapse fragility assessment using FEMA P-58-1 guidelines (ATC 2012). Such analysis will be useful in predicting loss for such structures making it easier for decision makers to quantify the performance of this building type.

Seismic performance assessment of RM components and systems had showed the ductile capabilities of such components under various seismic loading schemes (Priestley 1976, Priestley and Elder 1982, Fattal 1991, Leiva 1991, Shing et al. 1990-a and -b, Eikanas et al. 2003, Shedid 2009, Shedid et al 2010a, Vasconcelos and Lourenço 2009, Voon and Ingham 2006 and 2008, Haach et al. 2010, Banting and El-Dakhakhni 2012, El-Dakhakhni et al. 2013, Ahmadi et al. 2014). The assessment included field studies along with various testing protocols held by renowned research universities locally and internationally. Results from such tests are then used to evaluate crucial parameters such as strength, deformation capacities, stiffness degradation and energy dissipation.

A further step is to conduct a seismic risk assessment for masonry components and systems by developing what is known as collapse fragility curves. The new holistic view overseen by FEMA P-58-1 (ATC 2012) involves a performance assessment process. In this process, steps five and six involves determination of potential casualty, capital and economy losses as a function of structures and non-structural damage; computation of expected future losses as a function of intensity, structural and non-structural response and related damage. As stated by Nielson in his analytical fragility study for the Southeastern bridges in the US that realizing the usefulness of bridges fragility in assessing, managing and reducing the seismic risk had led to the development of several different fragility curve generation methodologies. The same concept can be used for RM shear wall systems. Until now two types of fragility curves exist, expert opinion based developed by ATC 1985, ATC 1991 and empirically based ones (Basoz and Kiremidjian 1997, Yamazaki et al. 1999, Shinozuka et al. 2003)

Collapse fragility assessment involves conducting incremental dynamic analysis (IDA) using several ground motions. The FEMA P-58-1 guidelines were followed in the selection of ground motions and in the development of the collapse fragility curves. A total of 15 simulated ground motion pairs were used to represent the design response spectrum for the target region, the City of Victoria, in the province of British Columbia which is known to be a highly seismic region. A typical RM shear wall (SW) building as shown in Fig. 1 was used to perform the study. The building plan shows four different types of RMSW with same overall aspect ratio spanning all over the building, three rectangular and one C-flanged. The seismic forces are transferred to the shear wall through a rigid floor slabs made of precast concrete hollow core slabs 20 cm in thickness. The building was designed according to the NBCC (NRC, 2010) and CSA S304-14 (CSA 2014) requirements. An analytical model was developed to discretize the RMSW building. Non-linear time history analysis (NLTHA) was then performed on the model to predict the dynamic behaviour of the building and to run the IDA procedure.

An analytical model for the shear wall system was developed to be used in the IDA. The building in Fig. 1 is simplified into stick 1D models representing the RMSW and rigid links representing the stiff floor slabs. The building can then be discretized as two shear walls system to measure the response of the building in the two orthogonal directions. Due to symmetry in the plan, half of the building can be modelled in each corresponding direction. The effect of torsional was also neglected for the same reason. The model was calibrated based on an experimental study on same walls specifications (Siyam 2014a). The walls were modelled using the displacement based beam column (DBBC) element obtained from OpenSees element library. The element object is a fiber based distributed plasticity model which takes into account flexure and axial interaction at the sectional level. The element takes both the masonry element and reinforcement non-linear behaviour into consideration and is based on linear interpolation of curvature, constant axial strain. After validating the model, IDA is performed using a suite of 15 ground motions pairs matching the response spectrum of Victoria region (Gonzales Heights). Results from the IDA are used to generate collapse fragility curve for the RMSW system. Such curves indicate the probability of collapse of such system types at various spectral accelerations. The following sections discuss further a brief summary of the experimental program and test results, details about the model used, analytical results and discussions, collapse fragility assessment and conclusions inferred from it.

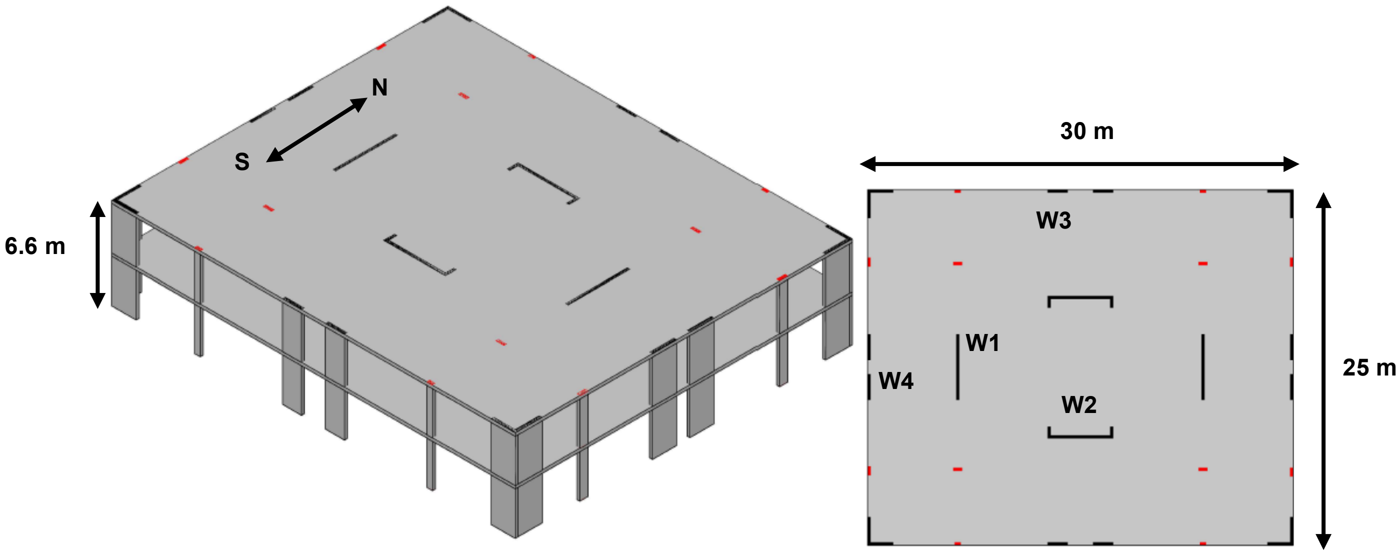


**Fig. 1 - (a) Archetype full-scale masonry building; (b) Plan Layout**

## 2. Summary of Experimental Program and Results

The experimental program focused on evaluating the seismic response of six, fully-grouted, RM shear walls subjected to reversed cycles of quasi-static loading. The walls were constructed using one-third scale blocks measuring 130 mm in length, 63 mm in width and 63 mm in height simulating the standard 190 mm stretcher units used throughout North America measuring 390 mm x 190 mm x 190 mm. Four of the walls (W1, W2, W3, and W4) have the same height and (overall) length but with different cross-section configurations, with Wall W1 being rectangular, Wall W2 being flanged and Walls W3 and W4 being coupled wall systems. In order to facilitate comparison Walls W5 and W6 are constructed and tested as they represented the individual components of the slab-coupled wall systems, Walls W3 and W4, respectively. In this respect, the response of two-linked Wall W5 can represent that of Wall W3 with a very flexible/weak slab coupling and the same for Wall W6 with respect to the coupled Wall W4.

Details of all the wall dimensions, aspect ratios and steel reinforcement ratios are presented in Table 1, in which the vertical and horizontal steel ratios, and the vertical and horizontal bar diameters are denoted by, ρv, ρh, dv and dh, respectively. Crack patterns of the walls at the final stages of testing are shown in Fig. 3. In general the experimental results showed that the ductile/special RM walls failed in a flexural manner reaching a displacement ductility level, at 20% ultimate strength degradation, between 5.4 and 7.6. The results also showed that RM shear walls detailed following the same shear wall classification, and having the same overall aspect ratio and reinforcement ratio, could experience significantly different force based (FB), displacement based (DB) and performance based (PB) seismic design parameters and therefore may have different ductility capacity parameters. Slab coupled walls shows better performance in terms ultimate drifts capacities reached, period elongation and equivalent viscous damping when compared to rectangular and flanged walls. Readers interested in further details of the test program, the experimental results, and the evaluated FBSD, DBSD and PBSD parameters, can refer to Siyam et al. (2014a and b).

**Table 1 Wall Details and Specification**

| Wall | Type | Height (mm) | Length (mm) | Aspect ratio | Vertical reinforcement | Horizontal reinforcement | | CSA Category | MSJC Category |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | $\rho_v$ (%) | $\rho_{h1}$ (%) | $\rho_{h2}$ (%) | | |
| W1 | Rectangular | 2160 | 1533 | 1.4 | 0.6 | 0.26 | 0.14 | Ductile | Special |
| W2 | Flanged | 2160 | 1533 | 1.4 | 0.6 | 0.26 | 0.14 | Ductile | Special |
| W3 | Rectangular | 2160 | 598 | 3.6 | 0.6 | 0.26 | 0.14 | Ductile | Special |
| W4 | Rectangular | 2160 | 465 | 4.6 | 0.6 | 0.26 | 0.14 | Ductile | Special |

# 3. ANALYTICAL STRUCTURAL MODELLING

## 3.1. Model Development

The SFRS of the building in Fig. 1 was modelled using OpenSees software. The model is a 1D fiber based macro model simulating the inelastic flexural behaviour of the walls and therefore the system (Ezzeledin et al. 2014). Macro modelling was sought to be a good choice since the author is interested in the global response of the system and the model will be used in a computationally intensive NLTHA. Moreover, macro-modelling of shear walls using fiber elements had shown considerable accuracy when modelling shear wall structures (Waugh and Sritharan 2010). The fiber element is a displacement based beam column elements that superimpose curvature and constant axial strain. Most fiber elements do not take the effect of shear deformations that occurs due to lateral load although recent experimental evidence shows the flexure and shear displacement are coupled for most of walls even for walls with aspect ratio as high as 3 (Massone and Wallace 2004). Since such walls exist in the building modelled, it is necessary to account for shear deformations in the model. The shear effect can be added manually or by introducing another element. In this model the shear deformations in the walls were aggregated using a uni-axial material model available in the OpenSees platform (Pinching 4 material) to prevent underestimation of top displacement. Global material response parameters for each wall type is estimated and inputted in the Pinching 4 material.

The system is discretized into six walls to represent the SFRS in the north-south direction of the building. Since the building is symmetrical in shape only half of the building was modelled. Each wall is discretized into seven elements in total, five elements are distributed along the first floor, one elements represent the top storey and a zero section element is added below wall-foundation interface. The rationale for this discretization is to capture the hinging mechanism that develops in the bottom storey and to model the tensile strain penetration that occurs below the footing level. As mentioned before, the model is 1D planar model where the bottom nodes were laterally restrained from motion. The fiber section is discretized by using masonry cells to represent the masonry area. The masonry is modelled using Concrete 07 material model which represent the simplified Chang and Manders 1994 concrete model. The fully grouted masonry is assumed to be in perfect bond with grout and mortar. The vertical reinforcement was modelled using Steel 02 material model which represent uni-axial Giuffre-Menegotto-Pinto steel material. Bond_SP01 material was added to model the effect tensile strain penetration of the vertical reinforcement.

The parameters used in each material model are explained in the subsequent section. It is important to state the assumptions taken by the developer in creating the model. The following assumptions were used in this fiber based element model. The walls are fixed at the bottom assuming rigid foundation system. Floor masses are assumed to be lumped at the floor levels at the centre of the walls. Ground motions are taken in the x and y directions and therefore only wall system parallel to direction of the earthquake are analyzed. The floor slab systems are modelled using rigid elastic elements with very high axial stiffness and therefore all the nodes within the same floor level displace equally.

## 4. METHODOLOGY AND DISCUSSIONS

### 4.1. Estimation of Ground Motions

A set of representative ground motion pairs were used to estimate the demand portion in the seismic fragility assessment. As mentioned earlier the FEMA guidelines were followed to choose a representative suite. Assatourians and Atkinson (2010) set of simulated strong ground motions were used to obtain Canadian ground motions since the region of interest is located in Victoria City.

The intensity based assessment was the selected procedure for ground motion selection and scaling as explained in FEMA P-58-1 (ATC 2012). It involves evaluating the performance of a SFRS for a user–selected acceleration response spectrum. The procedure requires first a selection of a target response spectrum to match the response spectra to it. Subsequently, a candidate suite of ground motions pairs is obtained from Assatourians and Atkinson (2010). The simulated suite contain linear response spectra with 5% damping from western regions at magnitudes ranging between 6.5 and 7.5, with soil condition C and focal distance varying between 10 to 100 km . Then the geomean spectrum, $S_{gm}$ given in Eq. 1 is computed for each ground motion pair over the period range between $T_{min}$ and $T_{max}$ where $S_x$ and $S_y$ are orthogonal components of spectral response acceleration at period T.

$$S_{gm}(T) = \sqrt{S_x(T) \times S_y(T)} \tag{1}$$

For this study the target response spectrum chosen is the City of Victoria, Gonzales heights with response spectrum shown in Fig. 2 to represent an area of high seismicity. The geomean response spectrums with shapes similar to the target response spectrum in the region of interest (Tmin $\leq$ T $\leq$ Tmaz ) are chosen. This makes a total of 15 ground motion pairs (3, 4 and 8) with characteristics corresponding to ground motions denoting west7c2, west6c2 and west6c1 respectively from Assatourians and Atkinson (2010) suite. The average matched geomean response spectrum (50th percentile) along with 16th and 84th percentile is also shown in Fig. 6. The final step involves scaling each ground motion pair by the ratio of $S_a(\overline{T})$ to the Sgm($\overline{T}$), where is $\overline{T}$ is the average fundamental period from the orthogonal directions (x and y) of the building. The scaled ground motion are then used in the IDA of the SFRS.

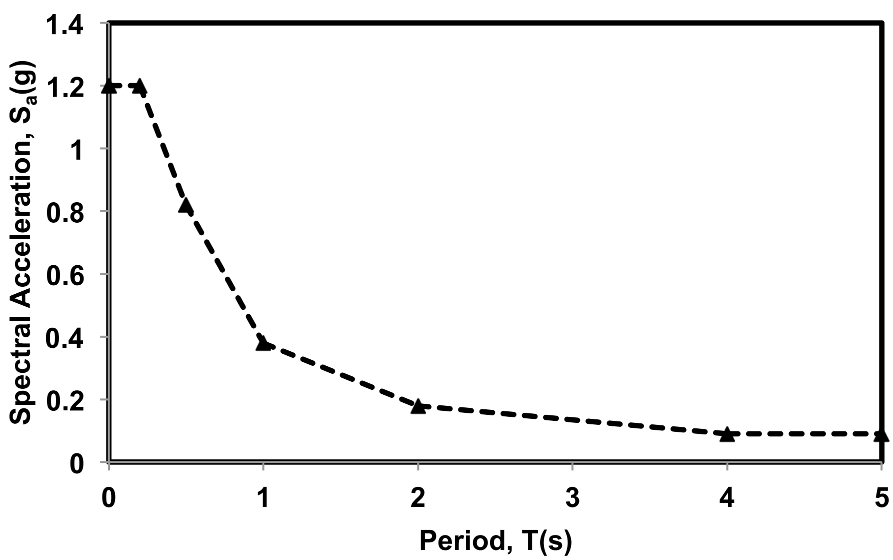


**Fig. 2 - Victoria City Gonzales Heights seismic hazard curve (NBCC 2010)**

### 4.2. Collapse Fragility Assessment

As stated in Baker 2014, fragility functions are derived using several approaches such as actual damage fragility from field observations or experimental testing (see Fig. 3), static structural analysis or by judgement (Kennedy and Ravindra 1984, Kim and Shinozuka 2004, Calvi et al. 2006; Villaverde 2007, Porter et al. 2007, Shafei et al. 2011).

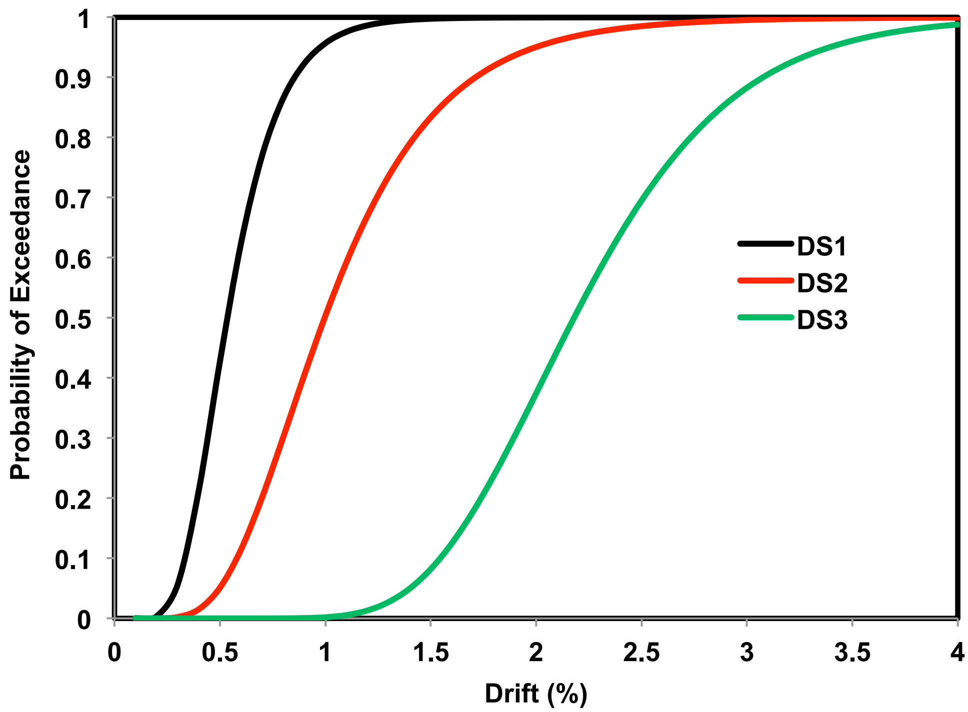


**Fig. 3 - Actual data fragility curves**

This study focus on analytical fragility curves developed from NTHLA. Recently, collapse fragility functions obtained from IDA results are becoming increasingly popular in structural assessment procedures (ATC 2012), since protection against collapse is a crucial objective in seismic design. Collapse fragility function can be described mathematically using Eq. 2, where P (C|*IM* =x), Φ, θ and β denotes the probability of collapse of structure given an intensity measure (*IM* = $S_a(T_1, 5\%)$) equal to x, the standard normal cumulative distribution function, the collapse median intensity(i.e. the IM level with 50% probability of collapse and standard deviation of intensity measure, respectively.

$$P(C \mid IM = x) = \Phi\left(\frac{\ln(x/\theta)}{\beta}\right) \tag{2}$$

The use of Eq. 2 implies log normality assumption of the ground motion which is a reasonable one as stated in other studies (Ibarra and Krawinkler 2005; Porter et al. 2007; Bradley and Dhakal 2008; Ghafory-Ashtiany et al. 2010; Eads et al. 2013). Collapse fragility curves are used for collapse safety assessment of building structures. Using such curves the probability of collapse at specific level of earthquake hazard can be predicted. The method is very sensitive according to the method of defining collapse in a structure. The next section will outline how to identify collapse.

### 4.3. Identifying Collapse State from IDA

Defining collapse of a SFRS is an essential step for creating collapse fragility curves. Collapse can be either local or global. This study focuses on global collapse of a structure. According to FEMA P-58 (ATC 2012), collapse is defined when sideway failure occur (lateral dynamic instability), loss of vertical load carrying capacity or exceedance of non-simulated failure criteria. Analytically collapse will cause numerical instability, unrealistic lateral drift response or demands exceeding the failure capacity of gravity load carrying components.

In this study, collapse is defined when certain damage measure reaches a specific capacity set by code provision in this case the NBCC 2010. Researchers have proposed several types of damage measures for wall components. Some involve displacement based measures such as maximum drift ratio and maximum top displacement while others prefer energy based measures that quantify the hysteretic energy at a given level of damage. Currently FEMA 58-1 uses maximum drift ratio to assess component and building performance level of damage of structural components. To be consistent with FEMA performance based assessment, the collapse point of $C_{DM}$ = 2.5% inter-storey drift is set to define the collapse of the RM system. Such value is extracted from the NBCC 2010 code (NRCC,2010) inter-storey drift limit set for collapse prevention

### 4.4. Collapse Fragility Curves and Fragility Fitting

The method of Moments estimator can be used to fit fragility curves from obtained results from IDA. The method involves estimating $\hat{\theta}$ and $\hat{\beta}$ by taking logarithms of each ground motion's IM values which was associated with onset of collapse point $C_{DM}$. Equations 3 and 4 can be utilized to compute such values.

$$\ln \hat{\theta} = \frac{1}{n} \sum_{i=1}^{n} \ln IM_i \qquad (3)$$

$$\hat{\beta} = \sqrt{\frac{1}{n-1} \sum_{i=1}^{n} (\ln(IM_i / \hat{\theta}))^2} \qquad (4)$$

Where n is the number of ground motions considered, and IMi defines the IM value associated with the onset of collapse for the ith ground motion. Such method basically represents the mean (which is equal to median in case of log normality) and standard deviation of normally distributed lnIM values (Baker 2014). The corresponding $\hat{\theta}$ and $\hat{\beta}$ using the $C_{DM}$ of 2.5% are 4.41g and 0.16. Moreover, the computed $\hat{\beta}$ value are 0.16 and 0.22. The collapse fragility curve in Fig. 4 using this method are shown as dotted lines.

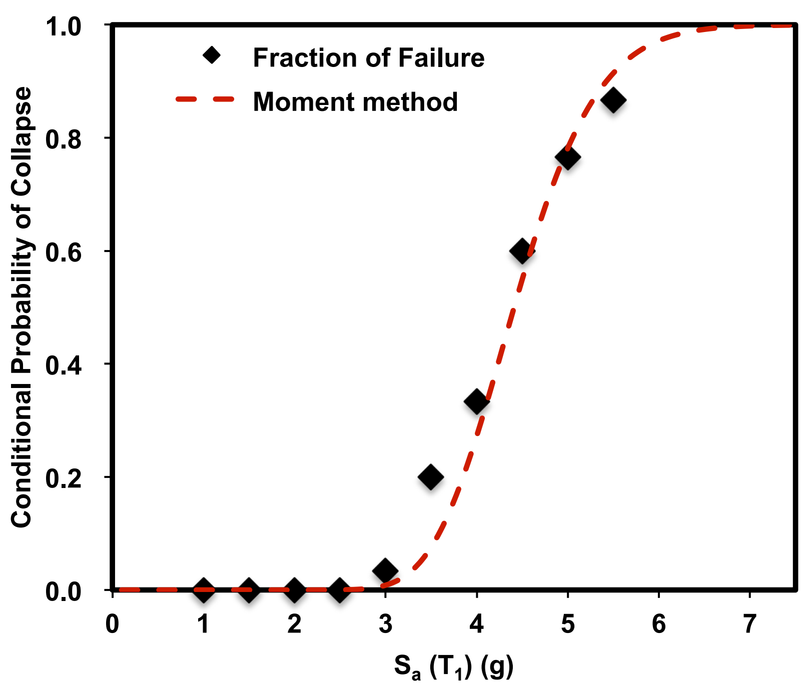


**Fig. 4 - Collapse Fragility Curve for N-S System**

## 5. Conclusions

The study presented a collapse fragility assessment of a two storey RM structure located in the region of Victoria in the province of British Columbia. The main SFRS is comprised of cantilever RM shear wall structures. An experimental study performed by the author presented results along with other seismic performance parameters that was used to validate the numerical model discussed in the study (Siyam 2014a and b). The seismic performance assessment process presented in the FEMA P-58-1 document (ATC 2012) was followed. The process involves selection and scaling of ground motion, a development of an analytical model that is used to perform NLTHA and development of fragility curves for a given structure to describe the probability of collapse given a seismic intensity, *IM*.

The analytical model is a 1D fiber based macro model simulating the inelastic flexural behaviour of the walls and therefore the system (Ezzeledin et al. 2014). Macro modelling was sought to be a good choice since the author is interested in the global response of the system and the model will be used in a computationally intensive NLTHA. Shear deformations in the walls were aggregated using a uni-axial material model available in the OpenSees platform (Pinching 4 material) to prevent underestimation of top displacement. The calibrated model seems to capture the non-linear response of RM shear walls with very good accuracy corresponding to walls W5 and W6 which have high aspect ratio. The numerical result for wall W1 matches the experimental ones for first four cycles and then the accuracy decreases for the last two cycles. The intensity based assessment was the selected procedure for ground motion selection and scaling as explained in FEMA P-58-1 (ATC 2012). The procedure requires first a selection of a target response spectrum to match the response spectra to it. The, a candidate suite of 15 ground motions pairs is obtained from Assatourians and Atkinson (2010) to provide Canadian design spectrum-consistent records for the City of Victoria. The simulated suite contain linear response spectra with 5% damping from western regions at magnitudes ranging between 6.5 and 7.5, with soil condition C and focal distance varying between 10 to 100 km . Seismic inelastic demands for the proposed masonry building in Fig. 1 were determined using Incremental Dynamic Analysis (IDA) procedure. The procedure provided a wealth of information about the non-linear behaviour of the RM SFRS under different seismic intensities. Collapse fragility assessment of the SFRS, using method of moment estimator fitting was computed from IDA results after defining collapse of a structure. The collapse of system was identified when maximum inter-storey drift reached a value of 2.5%. The collapse fragility analysis shows that RM SFRS designed under prescriptive CSA S304 (CSA 2014) code requirements presented therein, shows potential collapse capacity for high intensity earthquakes in a potentially high seismic region in Canada.

## 6. Acknowledgements

Financial support has been provided by the McMaster University Centre for Effective Design of Structures (CEDS) funded through the Ontario Research and Development Challenge Fund (ORDCF), as well as the Natural Sciences and Engineering Research Council (NSERC) of Canada. Provision of mason time by Ontario Masonry Contractors Association (OMCA) and Canada Masonry Design Centre is appreciated. The supply of the scaled blocks by the Canadian Concrete Masonry Producers Association (CCMPA) is gratefully acknowledged.

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