



13TH CANADIAN MASONRY SYMPOSIUM
HALIFAX, CANADA
JUNE 4TH – JUNE 7TH 2017



**SEISMIC PERFORMANCE AND HEIGHT LIMITS OF DUCTILE REINFORCED
MASONRY SHEAR WALL BUILDINGS WITH BOUNDARY ELEMENTS**

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ABSTRACT

The National Building Code of Canada, NBCC-15 has recently added a new Seismic Force Resisting System (SFERS) category, ductile shear walls, for RM structures designed and detailed according to Canadian Standards Association (CSA) S304-14. Ductile RM shear walls have special detailing requirements to ensure sufficient inelastic deformation capacity. Consequently, CSA S304-14 assigned higher ductility-related force modification factors to ductile RM shear walls compared to that of moderately ductile walls. However, NBCC-15 assigned the same building height limits for the ductile and moderately ductile walls. This study aims to assess (i.e. numerically) the seismic performance and collapse capacity of ductile RM buildings, having heights exceeding the code limit, built using RM shear walls with boundary elements as the SFERS. In this regards, a 12-story RM building located in a site in Vancouver, British Columbia was designed according to CSA S304-14 with ductile RM structural walls having confined boundary elements. The reference building had a total height exceeding the code specified limit. The seismic performance was evaluated using nonlinear pseudo-static pushover and Incremental Dynamic Analysis (IDA). The quantification of seismic performance and potential collapse capacity was executed using the procedure outlined in FEMA P695. The results revealed a superior seismic performance and high collapse capacity for the reference RM shear wall building. The seismic collapse capacity was found to be almost twice the acceptable values even at collapse probabilities as low as 5%. Therefore, these promising results indicate the possibility of using RM shear walls with well confined boundary elements as a SFERS in mid- and high-rise applications in regions with high seismic hazard. The findings of this study shed the light on the possibility of increasing the height limits assigned to ductile RM shear wall buildings.

KEYWORDS: *RM shear walls, boundary elements, ductility, nonlinear analysis, seismic response*

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INTRODUCTION

The capacity design philosophy introduced by Park and Paulay [1] requires selecting, designing and detailing a region or a component in the structure to develop a stable energy dissipation mechanism (i.e. without sudden loss in load carrying capacity). In addition, non-ductile (brittle) failure modes, such as shear failure, must be suppressed. Therefore, for the RM shear walls to be ductile in a seismic event, it should be capable of sustaining large reversible cycles of inelastic deformations without significant degradation in strength. Connecting confined boundary elements to the RM shear wall ends is one way of achieving this ductile response. RM shear walls constructed with boundary elements at the end zones demonstrate a significant enhancement in the wall curvature ductility compared to that of RM rectangular walls [2]. Consequently, RM shear walls with boundary elements present a potential Seismic Force Resisting System (SFRS) for mid- and high-rise RM buildings.

The addition of the confined boundary elements has a primary purpose of increasing the inelastic strain capacity of masonry. Thus, the ultimate compressive strain used in design can be significantly increased. In addition, the increased thickness of the added boundary stabilizes and reduces the depth of the compression zone. The effect of confinement on the ultimate strain has been reported by many researchers such as Mander et al. [3], and Saatcioglu and Razvi [4]. Moreover, as shown in Figure 1, the presence of the boundary elements result in postponing the onset of vertical reinforcement buckling as closely spaced lateral ties are provided to support the vertical bars. In addition, boundary elements reduce the post-peak strength degradation due to face-shell spalling as proved experimentally by Banting and El-Dakhakhni [5] and [6]. Therefore, as the boundary element confinement increases, the ultimate compressive strain and curvature of the wall are significantly enhanced. Along with the increase in ultimate curvature, boundary elements reduce curvature at the onset of yield. As a result, the section curvature ductility is improved resulting in a clear increase in the wall displacement ductility capacity.

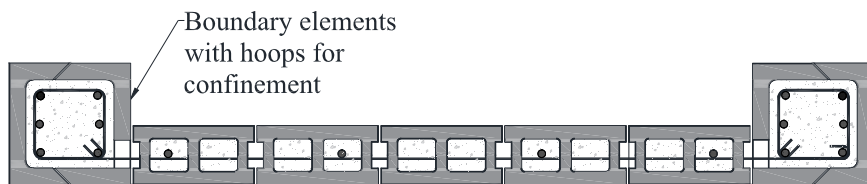


Figure 1: RM Shear Wall with Confined Boundary Elements

A new category of RM shear walls (i.e. ductile) was recently added to the National Building Code of Canada in 2015 [7] (i.e. indicated NBCC-15 hereafter). However, it is assigned the same building height limits of moderately ductile RM shear walls, as illustrated in Table 1. This could be attributed to the lack of knowledge about the seismic performance and collapse capacity of ductile RM shear walls. Several recent experimental and numerical studies of ductile RM masonry structural walls with boundary elements have highlighted its superior seismic performance and high collapse capacity such as Banting and El-Dakhakhni [6] and Ezzeldin et al. [8]. However, no

study up to the Authors' knowledge conducted a validation of the NBCC-15 specified building height limits for ductile RM shear wall buildings based on seismic collapse performance.

Table 1: NBCC-15 Building Height Limits

Height limit	
Seismicity [$I_E F_a S(0.2)$]	Ductile or Moderately Ductile RM Shear Walls
< 0.2	No Limit
≥ 0.2 to < 0.35	No Limit
≥ 0.35 to ≤ 0.75	60 m
> 0.75	40 m

The current study aims at evaluating the seismic performance and collapse capacity of RM ductile shear wall buildings with a focus on the specified building height limits and ductile RM shear walls with confined boundary elements. This was achieved by designing a 12-story RM building, exceeding the code specified height limit, and located in a region with high seismic hazard (i.e. Vancouver, British Columbia). Then, using an experimentally validated numerical model, nonlinear static and dynamic analyses are executed to evaluate the building seismic performance. The collapse capacity was evaluated and quantified using FEMA P695 [9] methodology. The resulting collapse capacity was compared against values recommended by FEMA P695 [9] for satisfactory seismic performance.

SELECTION AND DESIGN OF ARCHETYPE BUILDING

The selected archetype structure is a 12-story apartment building having the plan layout shown in Figure 2. A generic layout was chosen to represent masonry design for load bearing masonry buildings. The flooring system is composed of 150 mm thick precast prestressed Hollow Core Slabs (HCS) spanning for 7.5 m. The layout of HCS units was altered in plan to control the level of axial load on the walls such that it does not adversely impact its ductility and displacement capacity. The total typical floor height is 3.5m which allows for 2.7m clearance, 0.3m for flooring and 0.5m for overhanging (Mechanical, Electrical and Plumbing) services. Thus, the total building height is 42m. The building is located in Vancouver, British Columbia which has a high seismic hazard and the highest seismic risk in Canada. Soil class C was assumed for the building site. Wind loads are calculated based on NBCC-15 and, as expected, were not found to govern over seismic loads. The seismic hazard index, $I_E F_a S_a(0.2)$, for the building site in Vancouver is 0.848 which is higher than the code threshold value of 0.75. NBCC-15 specifies a maximum building height limit of 40m for ductile RM shear walls in regions with seismic hazard index greater than 0.75. Thus, the proposed reference building slightly exceeds the code assigned limit. The archetype building was designed and detailed in accordance to the special seismic design provisions provided in Canadian Standards Association (CSA) S304-14 clause 16. The structural system was a load bearing system with ductile RM shear walls resisting both vertical and lateral loads.

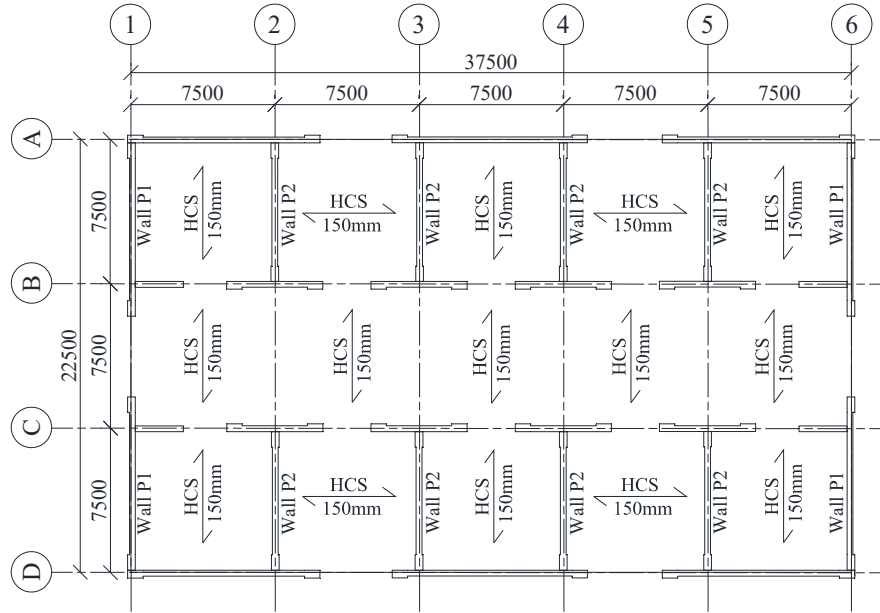


Figure 2: Typical Layout Plan

The shear walls were designed using standard concrete masonry stretcher blocks in the web and pilaster units in the boundary elements (see Figure 3). The walls were grouped and optimized every four floors. Summary of walls P1 and P2 design details (i.e. dimensions and reinforcement) is shown in Figure 3.

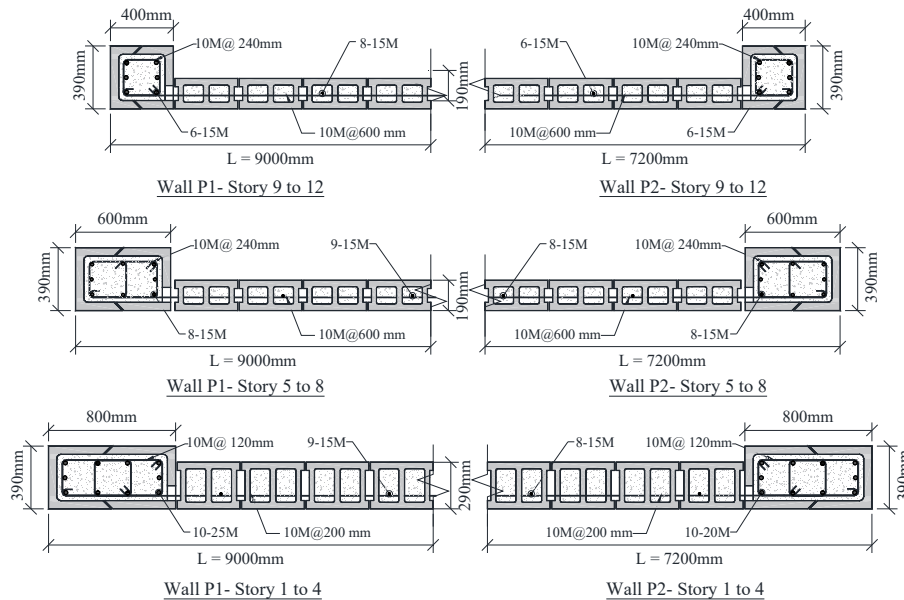


Figure 3: RM Walls Dimensions and Reinforcement Details

It can be seen that the boundary element size and web stretcher block width were reduced every four stories to optimize the design, whenever possible. For the first four stories, 290mm stretcher

blocks were used in the web whereas the boundary elements were made of two 390mm pilaster units to make the total length equal to 800mm. For the upper floors, the 190mm blocks were used for the webs and the boundary element length was reduced to 600mm then to 400mm.

The use of pilaster units in the boundary elements provides flexibility in using different numbers of vertical bars and spacing of transverse reinforcement. Thus, it is possible to provide closely spaced hoops to confine the grout core and increase the ultimate compressive strain (ϵ_{mu}) of grouted masonry. The increase in compressive strain due to confinement was calculated based on the provisions of CSA S304-14 clause 16.10 [10]. The spacing of hoops in first four floors was based on the requirements of buckling prevention or confinement ties. For upper floors, it was based on the requirements of lateral ties for reinforcement in compression as per CSA S304-14, clause 12.2.1 [10].

NUMERICAL MODELING VERIFICATION

For the building under study, a nonlinear numerical macro-model was developed on seismostruct program [11]. The RM shear walls were simulated using distributed inelasticity models with fiber sections. A displacement based beam-column element model was used with proper meshing based on a sensitivity analysis to reproduce the expected response. The nonlinear cyclic response of masonry was modelled using the uniaxial concrete material model proposed by Mander et al. [3] and the reinforcement nonlinear response was represented using the uniaxial stress-strain model derived by Menegotto and Pinto [12]. The nonlinear modeling approach was validated against existing experimental test results of RM shear walls. The experimental data was obtained from [5] and [6]. Summary of the walls' details is shown in Table 2. The walls used in the validation were end-confined reinforced masonry shear walls (i.e. walls with boundary elements). Comparison between experimental and numerical load-displacement response of the walls is shown in Figure 4. It can be seen from Figure 4 that the predicted hysteretic responses were in good agreement with the experimental results. It is observed that the proposed nonlinear model for RM shear walls is capable of accurately predicting the initial stiffness, yield strength, ultimate strength and drift capacity. The numerical predictions are within 10% of the experimentally measured values.

Table 2: Summary of Walls' Details Used for Numerical Model Validation

Wall	Reference	l_w (mm)	h (mm)	Aspect Ratio	Configuration	ρ_v (%)	ρ_h (%)	Axial Stress (MPa)
W 1	[5]	1803	3990	2.21	End Confined	0.56	0.30	0.45
W 2	[6]	1235	3990	3.23	End Confined	0.69	0.30	0.89

The following sections present the seismic response evaluation and quantification of the archetype building. The performance of the archetype building was assessed in the North-South (N-S) direction. Thus, only half of the building SFRS was modeled as the building is symmetrical.

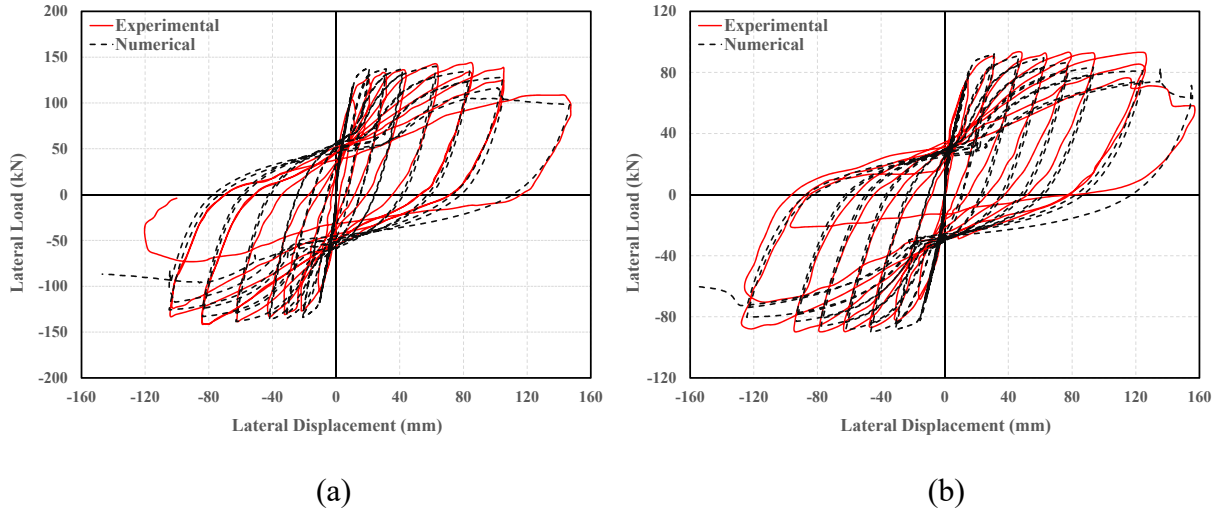


Figure 4: Experimental and Numerical Load-Displacement Response of: (a) End Confined Wall W1 [5]; (b) End Confined Wall W2 [6]

NONLINEAR PSEUDO-STATIC ANALYSIS

Nonlinear pseudo-static pushover analysis was performed to verify the nonlinear model, and evaluate the strength and deformation capacities of the archetype building. This analysis was executed following the approach outlined in FEMA P695 [9] which requires testing the reference building using a lateral force with vertical distribution that is proportional to the fundamental mode shape. Pushover analysis was conducted under the gravity loads expected during seismic events, as recommended by FEMA P695 [9]. Results were used to establish the capacity curve of the building SFRS in N-S direction as shown in Figure 5.

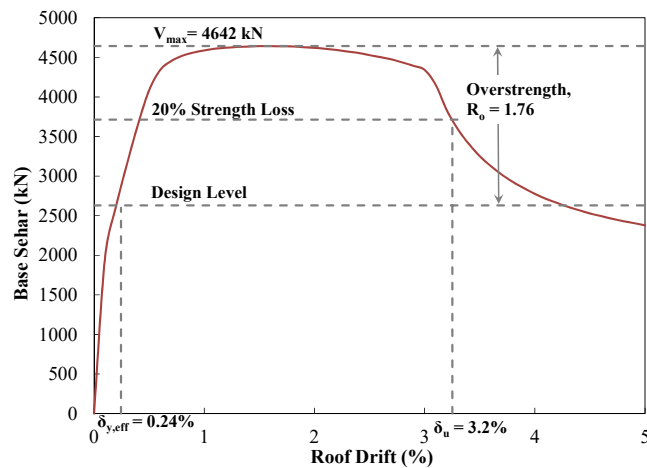


Figure 5: Pushover Curve for N-S SFRS

From the pushover curve, the weight-normalized base shear capacity of the building is 0.14, which corresponds to 1.6% roof drift ratio. The system overstrength (R_o) was calculated as the ratio between maximum base shear capacity (V_{max}) and design base shear. The calculated overstrength

value is 1.76, which is slightly higher than the code assumed dependable portion of reserve strength. This could be attributed to the conservative design approach adopted by most codes and the variation in sources that contributes to the structure overstrength.

Examination of Figure 5 confirms a clear ductile response for the reference RM shear wall building. The period-based ductility (μ_T) was calculated as defined in Equation (1) according to FEMA P695 [9].

$$\mu_T = \frac{\delta_u}{\delta_{y,eff}} \quad (1)$$

δ_u is the ultimate displacement at the roof corresponding to 20% degradation in ultimate capacity. The effective yield roof displacement ($\delta_{y,eff}$) was evaluated as per FEMA P695 by correlating the fundamental mode of vibration roof displacement of an idealized single degree of freedom system to the roof displacement of the building. The ductility (μ_T) was calculated to be greater than 8 for the archetype building and is attributed to the presence of the well detailed and confined boundary elements in the shear walls. The presence of boundary element significantly enhanced the ultimate curvature and reduced the curvature at the onset of vertical reinforcement yielding. This resulted in a clear improvement in the system displacement ductility. This enhancement in ductility of RM shear walls with confined boundary elements was also observed by others [8].

NONLINEAR DYNAMIC ANALYSES

A number of nonlinear response history analyses were performed to investigate the response of the reference building to ground motion excitations. A total of 24 random horizontal ground motions (12 Pairs) were selected for the response history analyses. This number is adequate to establish the record-to-record variability and calculate the median collapse intensity. The records were chosen from a database of processed time series generated for Canada by Assatourians and Atkinson [13]. They were selected and scaled to match the design response spectrum of Vancouver following the intensity-based performance assessment approach of FEMA P58-1 [14]. The building was subjected to each of the scaled ground motion records with multiply increased intensities. This dynamic analysis approach is termed as Incremental Dynamic Analysis (IDA) and is described in detail by Vamvatsikos and Cornell [15]. IDA allows capturing the behavior from the elastic range until collapse is recognized in the structure. The IDA results were used to construct IDA curves. They are typically plots relating an Engineering Demand Parameter (EDP) to an Intensity Measure (IM).

In this study, Peak interstory drift ratios are selected as the EDPs to represent the global performance of the building. This is consistent with the main objective of investigating the specified code (NBCC-15) limits on building heights. For the IM, 5% damped spectral accelerations at the building's natural period are selected ($S(T_1, 5\%/g)$). This allows representative and direct application of FEMA P695 [9] methodology for the seismic performance and collapse capacity evaluation. Figure 6 depicts the IDA curves of the reference building in the N-S direction.

Each point on the IDA curve typically represents the building response to a ground motion record at a certain intensity scaling factor.

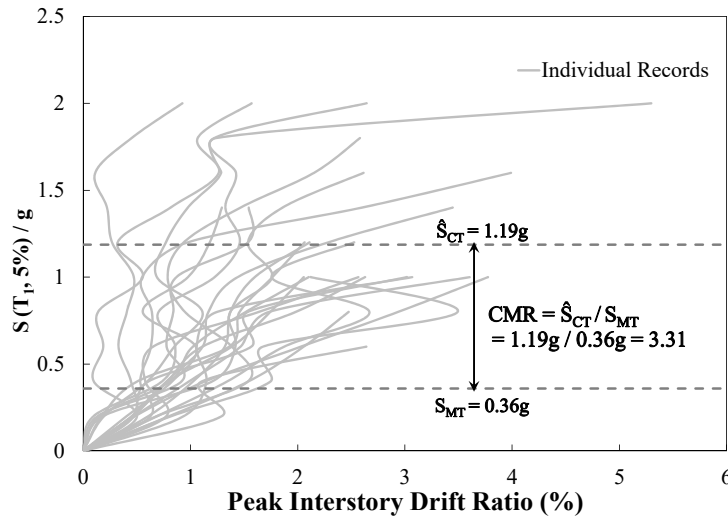


Figure 6: IDA Curves of the SFRS in N-S Direction

COLLAPSE CAPACITY ASSESSMENT

For assessment of the building seismic collapse performance based on FEMA P695 [9], it is required to estimate the median collapse intensity (\hat{S}_{CT}). The median collapse intensity is defined as the 5% damped spectral acceleration at the structure's natural period which corresponds to 50% probability of collapse (i.e. when 50% of the records causes collapse). Based on FEMA P-58 [14] and P695 [9], seismic collapse could be either simulated or non-simulated collapse. Simulated seismic collapse can be realized due to dynamic instability or a sidesway collapse mechanism resulting from large lateral displacements. On the other hand, non-simulated seismic collapse is usually when a specific component limit state defined by the user is exceeded. For the objective of this study, it is most appropriate to assume that collapse is realized when there is a sidesway collapse mechanism being developed in the archetype building. Therefore, seismic collapse was assumed when the peak interstory drift exceeds the NBCC-15 limit of 2.5% for buildings with normal importance. This value achieves the code (i.e. NBCC-15) primary objective of protecting the life and safety of building occupants as the building is subjected to severe ground shaking, and thus suitable to assess the code building height limits for ductile RM shear wall buildings.

As illustrated in Figure 6, the calculated median collapse intensity (\hat{S}_{CT}) was 1.19g. This value was compared to the Maximum Considered Earthquake (MCE) level spectral acceleration at the natural period of the building (S_{MT}) to calculate the Collapse Margin Ratio (CMR). The CMR of the reference building was 3.31 which reflects a significantly high margin of safety against simulated seismic collapse.

IDA results were used to calculate the collapse probabilities at the different IM increments based on the previous definition of collapse. Then, those results were fitted using a Cumulative

Distribution Function (CDF) assuming lognormal distribution of the IM increments causing collapse. The parameters of the fragility fitting function, the median (θ) and dispersion (β), were estimated following the maximum likelihood method [16]. In this method, the mean and the dispersion are estimated so that the resulting CDF (Φ) has the maximum likelihood of reproducing the observed analytical data points. The collapse fragility function relates the probability of collapse to any given IM. The observed, fitted and adjusted collapse fragility curves are shown in Figure 7.

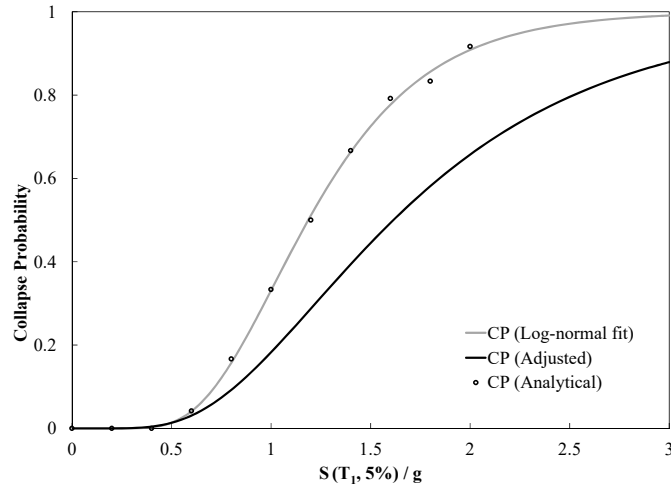


Figure 7: Reference Building SFRS Collapse Fragility Curves in N-S Direction

The IDA results were adjusted to account for the various uncertainty sources using FEMA P695 methodology to calculate the Adjusted Collapse Margin Ratio (ACMR). This include the records' spectral shape and total system collapse uncertainty. To consider the variations in records' frequency content and the elongation in the structure's natural period prior collapse, the calculated CMR was multiplied by the Spectral Shape Factor (SSF). SSF was estimated from Table 7-1 in FEMA P695 depending on the natural period and period-based ductility factor (μ_T) of the building. For the studied building, the SSF was 1.36 which resulted in an ACMR of 4.48. It is evident in Figure 7 that the resulting collapse fragility curve of the reference building is relatively flat, representing low increase in collapse probabilities as the IMs increase. The calculated ACMR was compared against the acceptable values proposed by FEMA P695. The acceptable ACMR values were estimated from Table 7-3 in FEMA P695 based on the total system collapse uncertainty (β_{TOT}) which was calculated using Equation (2).

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \quad (2)$$

The total system collapse uncertainty was calculated based on the record-to-record variability (β_{RTR}), design requirements uncertainty (β_{DR}), test data related uncertainty (β_{TD}), and modeling related uncertainty (β_{MDL}). Using the calculated total system collapse uncertainty (β_{RTR}) of 0.529, the acceptable ACMRs were 2.39, 1.97 and 1.56 for 5%, 10% and 20% collapse probabilities,

respectively. In Table 3, it is clear that the computed ACMR of the ductile RM shear walls with confined boundary elements is significantly higher than the acceptable values. The calculated ACMR is 88%, 127% and 186% higher than acceptable values at 5%, 10% and 20% collapse probabilities at MCE, respectively. This indicates that ductile RM shear wall buildings that are well designed and detailed in accordance to CSA S304-14 have significantly high reserve capacity against simulated seismic collapse.

Table 3: Seismic Collapse Capacity Acceptance Check

Computed ACMR	Collapse Probability	Acceptable ACMR	Check	% Difference
4.48	5%	2.39	Pass	88%
	10%	1.97	Pass	127%
	20%	1.56	Pass	186%

CONCLUSIONS

Nonlinear pushover analysis results indicated a favorable response for ductile RM shear wall buildings with boundary elements. The resulting response from monotonic loading indicated reasonable strength and high deformation capacities. IDA results showed a favorable seismic performance for the archetype building characterized by a high median collapse intensity (\hat{S}_{CT}). Developed collapse fragility curves confirmed the superior seismic performance and the high reserve collapse capacity at MCE level. This was evident in the high ACMR of 4.48 when compared to the acceptable ACMR at 20% collapse probability of 1.56. The system ACMR is 188%, 127% and 86% higher than acceptable values at 20%, 10% and 5% collapse probabilities at MCE, respectively. This reflects that the reference building had a high reserve of capacity against simulated seismic collapse. Therefore, these promising results demonstrate the possibility of relaxing the building height limits for ductile RM shear walls. The presented results are part of an ongoing research program investigating the building height limits for ductile RM shear walls with boundary elements built in varying seismic hazards locations.

ACKNOWLEDGEMENTS

The Authors acknowledge the support from the Natural Science and Engineering Research Council of Canada (NSERC), l'Association des Entrepreneurs en Maçonnerie du Québec (AEMQ), the Canadian Concrete Masonry Producers Association (CCMPA) and the Canadian Masonry Design Centre (CMDC).

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