## Vulnerability Assessment of Seismic Induced Out-of-Plane Failure of Unreinforced Masonry Wall Buildings

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Vulnerability Assessment of Seismic Induced Out-of-Plane Failure of Unreinforced Masonry Wall Buildings

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ABSTRACT:

Damage to unreinforced masonry (URM) buildings from earthquake shaking is often caused by out-of-plane failure of walls. This is particularly relevant to the majority of URM buildings in Eastern Canada that were constructed prior to the introduction of seismic design prescriptions. Seismic vulnerability assessment of this type of failure is therefore an essential step towards seismic risk mitigation. This paper presents a simplified procedure for seismic vulnerability assessment of out-of-plane failure of URM wall buildings. The procedure includes the development of an equivalent single degree of freedom model of the wall with a characteristic force-deformation capacity curve. The capacity curve is convolved with displacement response spectrum to predict the displacement demand. The predicted displacement demand is compared to displacement thresholds criteria corresponding to the initiation of each damage state. The procedure is applied to an inventory of URM buildings in Montreal and the corresponding probability of out-of-plane damage is evaluated.

Keywords: Seismic vulnerability assessment, fragility analysis, unreinforced masonry, out-of-plane damage.
1 INTRODUCTION

Post-earthquake damage reports showed that unreinforced masonry (URM) buildings are among the most vulnerable structures to earthquakes (Coburn and Spence 2002; Doherty et al. 2002). Inspection reports following the 2010 Christchurch earthquake with a magnitude of 6.3 indicated that a large proportion of damages to URM building were attributed to out-of-plane failures (Ingham and Griffith 2011). The most seismically vulnerable URM components are: parapets, chimneys, gables, brick veneers and unattached walls sensitive to out-of-plane failure. Recent studies have shown that the out-of-plane vulnerability of URM components or walls is associated with the increase in displacement demand; therefore, displacement based assessment procedures were developed to model the out-of-plane displacement capacity response of URM walls' (e.g. Doherty et al. 2002; Griffith et al. 2003; Derakhshan et al. 2013). Moreover, seismic analysis procedures have been developed in Italy for out-of-plane collapse mechanisms based on research conducted on equilibrium limit analysis and the identification of collapse displacement limit state (De Felice and Giannini 2001; D’Ayala and Speranza 2003; Sorrentino et al. 2008; Lagomarsino and Resemini 2009; Magenes and Penna 2011). In displacement based analysis, displacement demands are compared to displacement capacity limit states to evaluate the probability of reaching or exceeding specific damage states which are typically defined as fragility functions (Lumantarna et al. 2006; Antunez et al. 2015). Fragility functions are particularly useful for risk-informed decision making, for retrofit and risk mitigation planning (Coburn and Spence 2002; Abo-El-Ezz et al. 2013). Fragility functions can be developed based on damage data derived from post-earthquake surveys, expert opinion, analytical modelling or...
combinations of these (Jeong and Elnashai 2007). In regions of high seismicity, the
availability of post-earthquake damage data allows for the development of observed
fragility functions (Coburn and Spence 2002). On the other hand, in regions with limited
recorded damage data, such as Eastern Canada, risk assessment relies mainly on the
development of analytical fragility functions. Therefore, there is a need to develop
analytical procedures for seismic fragility analysis of out-of-plane failure of URM
buildings that reflect the generic construction characteristics for the considered study
area.

In Eastern Canada, a large proportion of residential buildings are either URM structures
with load bearing walls or wood framing structures with URM components such as brick
veneers or chimneys (Nollet et al. 2016; Abo-El-Ezz et al. 2015). A majority of these
buildings were built before the introduction of seismic design standards and codes and
their response to future seismic events, even of moderate intensity, is a concern. The
main objective of this study is to conduct quantitative assessment of seismic performance
and vulnerability of representative buildings located in Montreal and having URM load
bearing walls prone to out-of-plane failure. In order to achieve this objective, fragility
functions that correlate the probability of damage to the seismic intensity measure (e.g.
peak ground acceleration, PGA) are developed. The study evaluates the structural
characterisation of existing URM load bearing buildings in Montreal region to identify
typical facade properties that are susceptible to out-of-plane failure. A simplified
probabilistic nonlinear static based procedure is developed to evaluate the seismic
demand using an equivalent Single Degree of Freedom (ESDOF) model. The seismic
demands are then compared to displacement thresholds criteria proposed by the authors
to develop the corresponding fragility functions for different damage states. The
developed analytical fragility functions are then used to evaluate the out-of-plane seismic
vulnerability for URM buildings. The evaluation is conducted for the seismic hazard
corresponding to the design level ground motion with 2% probability of exceedance in 50
years as defined in the National Building Code of Canada (NBCC) (NRCC 2010), and for
ground motion with 10% probability of exceedance in 50 years obtained from the seismic
hazard calculator website (www.EarthquakesCanada.ca). An important feature of the
developed fragility analysis procedure is the simplicity and reliability of its application to
a large number of buildings within a region with reduced computational time. To the
author’s knowledge, this study presents one of the first attempts to propose and validate a
simplified step-by-step procedure for the development of fragility functions of out-of-
plane loaded URM walls using site-specific geometrical and material parameters to be
used for seismic risk assessment studies at a regional scale. In order to evaluate the
reliability of the developed fragility functions, a comparative evaluation of the developed
analytical fragility functions of out-of-plane failure with existing fragility functions for
URM buildings is presented. Emphasis is put on out-of-plane collapse of URM walls
since it is one of the main causes of casualties.

2 VULNERABILITY ASSESSMENT PROCEDURE

Out-of-plane vulnerability assessment is conducted using analytical fragility functions.
These functions are typically given in the form of lognormal distribution of the
probability of being in or exceeding a given damage state for a given intensity measure
(IM) (e.g. PGA). The conditional probability of attaining a particular damage state (DSi),
given the IM, is defined in Equations 1 and 2 (Kircher et al. 1997).
\[ P[DS \mid IM] = \Phi \left[ \frac{1}{\beta_{DS}} \ln \left( \frac{IM}{IM_{DS}} \right) \right] \]  

\[ \beta_{DS} = \sqrt{\beta_{C}^2 + \beta_{D}^2 + \beta_{T}^2} \]

Where \( IM_{DS} \) is median value of the IM at which the building reaches the threshold of damage state DS, and \( \Phi \) is standard normal cumulative distribution function. \( \beta_{DS} \) is standard deviation of the natural logarithm of the IM for damage state DS. The standard deviation of the fragility function represents the variability in the prediction of damage given an IM. The variability in damage prediction is composed of three components: the variability in the seismic demand \( \beta_{D} \), the variability in the seismic capacity corresponding to damage state \( \beta_{C} \) and the variability in the threshold of the damage state \( \beta_{T} \). Default values of the standard deviations can be assumed in order to capture in an approximate manner the variability in damage assessment of building as an alternative to conducting time-consuming nonlinear dynamic analyses (FEMA 2003; FEMA P-58 2012; D’Ayala et al. 2015; Porter et al. 2015). In this study, default values of the standard deviations are assumed based on the recommended values in Hazus Advanced Engineering Building Model (FEMA.2003) where \( \beta_{C} = 0.25, \beta_{D} = 0.50 \) and \( \beta_{T} = 0.20 \) which gives a total standard deviation of \( \beta_{DS} = 0.6 \). These Hazus values were developed based on a combination of experimental results, earthquake damage observations and expert opinion. The assumed standard deviation (0.6) in this study provides an acceptable estimate for rapid generation of the fragility functions for a portfolio of buildings for regional scale studies.

For a given URM wall susceptible to out-of-plane failure, the procedure for the development of fragility functions can be outlined as follows:
1) Development of capacity curve: The displacement capacity of an URM wall can be represented by a tri-linear curve for the ESDOF model.

2) Prediction of displacement demand: The displacement demand can be estimated using an equivalent linear ESDOF model with an equivalent period and viscous damping ratio. The seismic displacement demand of the model is obtained using equivalent linear response spectrum analysis for increasing levels of a selected IM (e.g. PGA). The application of the ESDOF model showed reasonably good approximation of the seismic displacement demands when compared to experimental results obtained from shake table tests (Doherty et al. 2002; Houalard et al. 2015). In the context of seismic assessment of a large population of buildings, the use of ESDOF models is a reasonable and accepted assumption as it presents a less-time consuming alternative to computationally expensive detailed finite element models for masonry.

3) Development of damage states fragility functions: The displacement capacities to reach different damage states are identified (e.g. cracking, collapse). Then, a convolution of the displacement demand and capacity models is performed in order to develop fragility functions corresponding to the probability of exceedance of different damage states in terms of the selected IM.

2.1 Capacity curve for URM wall

This section presents the development of the capacity curve of the ESDOF model for the simulation of the lateral force-deformation corresponding to out-of-plane response of URM walls. The behaviour of URM walls subjected to seismic excitation can be modeled by rigid blocks separated by cracked section (Doherty et al. 2002). Moreover,
mechanisms of damages depend on several parameters such as geometric properties, boundary conditions, location of the element and characteristics of openings. For simplicity of analyses it is often assumed that walls are supported only along their top and bottom edges, so that wall failure is generally in the form of a horizontal crack located above the wall mid-height. These assumptions lead to a lower bound result and have been adopted in the presented model, but it is also acknowledged that existing walls have supports along their vertical edges to orthogonal walls (Derakhshan et al. 2014). These edge restraints may resist rotation. Such rotational restraint is often neglected because of uncertainties in modeling such action (Abrams et al. 2017). This assumption is acceptable in the context of seismic risk assessment of large number of buildings, as it allows to capture the initiation of out-of-plane damage in the most vulnerable elements. It provides rapid estimate of the fragility functions with limited number of input parameters that are typically available for buildings.

Displacement capacity is influenced by the wall thickness (t) and its aspect ratio (h/t), while the constraint capacity depends on boundary conditions (Doherty et al. 2002). In order to facilitate the evaluation of the out-of-plane vulnerability of URM walls, Doherty et al. (2002) suggested using a simple equivalent parapet model reflecting the different configurations and boundary conditions of walls, as illustrated in Figure 1. The equivalent parapet model is defined by equivalent thickness (t_{equiv}) and height (h_{equiv}) (as defined in Figure 1) that depend on the boundary conditions of the wall and the overburden ratio acting on it (\psi) which is defined as the ratio of overburden weight and self-weight of the wall. This parapet wall can then be simplified into an ESDOF model. As shown in Figure 1, two configurations of URM walls characterised by different overburden and cracking at mid-height are considered in this study: (a) rigid load bearing

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simply supported wall with slab boundary condition at the top, and (b) rigid load bearing simply supported wall with a timber bearer boundary condition so the top reaction is centered.

The observed response of cracked out-of-plane wall subjected to out-of-plane loading is curvilinear (Doherty et al. (2002). It can be predicted through the equivalent tri-linear capacity model and the classical rigid body bilinear equilibrium model shown in Figure 2. Four parameters are used to draw the tri-linear capacity model: the wall instability displacement $\Delta_{ins}$ (Equation 3), the empirical displacement values $\Delta_1$ (Equation 4), and $\Delta_2$ (Equation 5), and the maximum force $F_i$ (Equation 6). Equations 3 to 8 are expressed in terms of the equivalent thickness $t_{equiv}$ and equivalent height $h_{equiv}$ for the equivalent parapet model shown in Figure 1. $\Delta_1$ is defined as an empirical displacement at which wall’s force–displacement relation reaches its maximum strength ($F_{max}$ in Figure 2); $\Delta_2$ is defined as an empirical displacement at which wall’s maximum force plateau intersects with the rigid body bilinear model; $\Delta_{ins}$ is defined as the failure displacement at which the wall becomes unstable and $F_i$ (Equation 6) is defined as the actual wall lateral strength which is calculated as a function of the rigid body lateral strength $F_o$ (Equation 7). The effective mass was considered as equal to $(3/4)$ of the mass of the wall ($M$) for the computation of the lateral strength. The reader is referred to (Doherty et al. (2002); Derakhshan et al. (2013) for detailed derivation of the listed equations. Experimental studies were conducted to define the empirical displacements $\Delta_1$ and $\Delta_2$ as a function of $\Delta_{ins}$ (Doherty et al. 2002; Griffith et al. 2004; Derakhshan et al. 2013). Derakhshan et al. (2013) observed that the wall instability displacement $\Delta_{ins}$ and displacement values $\Delta_1$, and $\Delta_2$ are sensitive to the crack height ratio ($\beta$), the overburden ratio ($\psi$) and the mortar compressive strength ($f'_j$). The crack height ratio ($\beta$) is defined as the ratio of the height
of the location of the pivot points of the crack that forms in the wall to the total wall height (shown in Figure 1 as the dotted line in the wall). For simply supported walls, $\beta$ is assumed equal to 0.5 (mid-height crack). The overburden ratio ($\psi = P_o/W$) is defined as the ratio of the axial load from the floor ($P_o$) applied on the top of the wall to the self-weight of the wall ($W$).

The tri-linear model considers the influence of finite masonry compressive strength on the lateral strength of the wall through an empirical parameter called PMR\textsubscript{emp} (Percentage of Maximum rigid Resistance) (Equation 8). The PMR\textsubscript{emp} is defined as the ratio of the lateral strength achievable by a real URM wall ($F_{max}$), as shown in Figure 2, to the bilinear rigid block strength assuming infinite masonry compression strength ($F_o$). This ratio is always less than unity due to the finite masonry compressive strength. Derakhshan et al. (2013) derived a theoretical mechanics-based equation for the PMR and observed that the experimentally obtained (PMR\textsubscript{emp}) is equal on average to 0.83 times the theoretical PMR due to the roundedness of wall corners and prior masonry crushing at pivotal points, which were not accounted for in the theoretical mechanics-based formulation. The idealized lateral strength of the capacity curve ($F_1$) is assumed equal to 0.9 $F_{max}$ based on experimental calibration. The reader is referred to Derakhshan et al. (2013) for the full derivation of the theoretical and experimental calibration of the PMR\textsubscript{emp}.

\begin{align*}
\Delta_{ins} &= \frac{2}{3} t_{equiv} \quad (3) \\
\Delta_1 &= 0.04 \Delta_{ins} \quad (4) \\
\Delta_2 &= (1 - 0.009 \text{ PMR}_{\text{emp}}) \Delta_{ins} \quad (5)
\end{align*}
\[ F_i = 0.9 \left( PMR_{emp} \cdot F_0 \right) \]  \hspace{1cm} (6)

\[ F_0 = \frac{M_c \cdot g \cdot t_{equiv}}{h_{equiv}} = \frac{3 M_c \cdot g}{4} \left( \frac{t_{equiv}}{h_{equiv}} \right) \]  \hspace{1cm} (7)

\[ PMR_{emp} \% = 83 \left[ 1 - \frac{\rho \cdot h \cdot g}{0.85 \cdot f_j^i} \cdot \left( \psi + \frac{(1 - \beta)(2\psi + 2 - \beta)}{2(1 - \beta) + (2 - \beta)\psi} \right) \right] \]  \hspace{1cm} (8)

2.2 Damage states

The relatively good statistical correlation that was observed between the seismic-induced maximal displacement of a structure and the extent of structural damage contributed to the development of modern performance-based seismic assessment methods. These methods consist in evaluating the structure specific deformation capacity and earthquake-induced displacement demand (Ruiz-Garcia and Negrete 2009). Therefore, seismic performance can be assessed using wall displacement, related to the wall’s physical damage state following the ground shaking. In order to evaluate the seismic out-of-plane performance of URM walls, it is of interest to evaluate the probability of exceedance of displacement thresholds corresponding to different damage states. The most commonly identified damage state for out-of-plane response of URM walls is the threshold of wall collapse when the displacement demand exceeds the wall instability displacement, \( \Delta_{ins} \) (Restrepo-Velez and Magenes 2004; Lumantarana et al. 2006; Borzi et al. 2008). Krawinkler et al. (2012) identified two damage states for URM parapets and chimneys. In the first damage state damage is apparent (i.e. visible cracking, sliding of the chimney/parapet), likely resulting in a Yellow Tag defined as limited entry and restricted use to the building (ATC 2005) with area unsafe, and requires removal or replacement of that portion of masonry above the crack. The second damage state captures all toppling
damage that has potential for human injury. Four damage states are identified in the
FEMA-306 report (FEMA 1998) for rigid-body rocking motion of URM walls including:
insignificant, moderate, heavy and extreme damage. The insignificant damage state
represents hairline cracks at floor/roof lines and mid-height of stories. The moderate
damage represents cracks at floor/roof lines and mid-height of stories with mortar
spalling to full depth of joint and possibly out-of-plane offsets along cracks. The heavy
damage state represents spalling of units along crack plane with out-of-plane offsets
along cracks and significant crushing/spalling of bricks at crack locations. The extreme
damage state represents a wall with threatened vertical-load-carrying ability, significant
out-of-plane movement at top and bottom of the wall and significant crushing/spalling of
bricks at crack locations. These damage states are only described in terms of qualitative
characterisation without identifying associated displacement thresholds. The FEMA-306
report (FEMA 1998) stated that “as rocking increases, the mortar and masonry units at
the crack locations can be degraded, and residual offsets can occur at the crack planes.
The ultimate limit state is that the walls rock too far and overturn”. Lumantarana et al.
(2006) considered three displacement thresholds for minor, moderate and collapse
damage states. The minor damage threshold at which the wall is expected to undergo first
cracking was associated with a wall displacement of 5mm. The displacement limit at
moderate damage was arbitrarily defined as equivalent to half of the wall thickness. URM
walls subject to displacement exceeding this limit are expected to have a fully developed
crack pattern that forms a collapse mechanism. The displacement limit at collapse was
defined at the wall thickness. Based on the interpretation of the above references and the
authors engineering judgment, four damage states with three corresponding displacement
thresholds are considered in this study: insignificant (DS0), moderate (DS1),
heavy/extreme (DS2) and collapse (DS3) (Table 1). The displacement threshold for the
moderate damage is identified at a displacement value equal to $\Delta_f$ at which the wall
reaches its maximum force capacity (Figure 2). From this state, rocking response of the
wall starts with no strength degradation. The displacement threshold for the heavy
damage is identified at a displacement value equal to $\Delta_2$ after which a reduction in lateral
strength of the wall is observed. Finally, the displacement threshold for the onset of
collapse is identified at a displacement value equal to $\Delta_{ins}$, corresponding to overturning
of the wall.

2.3 Displacement demand prediction

The development of fragility functions requires a seismic demand model providing a
prediction of the displacement response for increasing level of ground motion intensity.
In order to evaluate the displacement demand for the out-of-plane response of URM
walls, the equivalent linear method is applied in this study (Doherty et al. 2002). The
spectral displacement of an equivalent linear ESDOF with an equivalent period ($T_e$) and
viscous damping ratio ($\zeta$) is compared to a given linear response spectrum to estimate the
displacement demand. Griffith et al. (2003) evaluated the mean difference between the
equivalent linear method predictions of the displacement demands and the results of
analytical modelling of the out-of-plane response of URM walls using nonlinear time
history analyses. The nonlinear time history analysis was conducted on multiple URM
wall configurations idealised as nonlinear spring element in the software FEAP [Taylor,
2000]. The force-deformation relationship for the nonlinear spring element is based on
the Doherty tri-linear model. The following observations were reported: (1) the
application of the equivalent period ($T_1$) (Figure 3a, Equation 9) and 5% damping ratio
showed the lowest mean difference in predicting the displacements when the
displacement demands were less than 50% of the instability displacement; (2) on the
other hand, the application of the equivalent period (T₂) (Figure 3a, Equation 9) and 5%
damping ratio showed the lowest mean difference in predicting the displacements when
the displacement demands were greater than 50% of the instability displacement.

\[ T_{1,2} = 2\pi \sqrt{\frac{M_e}{K_{(1,2)}}} = 2\pi \sqrt{\frac{0.75M}{K_{(1,2)}}} = 2\pi \sqrt{\frac{0.75 \times \rho_m \times h \times t \times L}{F_1/\Delta_{1,2}}} \] (9)

In this study, two seismic demand models with two corresponding displacement
thresholds were developed. The first model applies the equivalent period (T₁) for
comparison with the displacement thresholds that are less than 50% of \( \Delta_{\text{ins}} \), denoted \( \Delta_1 \) for
the moderate damage. The second model applies the equivalent period (T₂) for
comparison with the displacement thresholds that are greater than 50% of \( \Delta_{\text{ins}} \) (i.e. the
displacement threshold for the onset of collapse \( \Delta_{\text{ins}} \)), denoted \( \Delta_2 \) for the heavy damage.

The procedure to develop the seismic demand models is as follows:

1) Define the tri-linear capacity model of the URM wall using Equations 3 to 8 with
the geometric characteristics of the wall and compute the equivalent fundamental
periods (T₁ and T₂) of the ESDOF model using Equation 9;

2) For a given response spectrum anchored to a specific level of an IM (e.g. PGA),
determine the wall displacement (\( \Delta_w \)) (Equation 10) corresponding to the spectral
displacement, \( S_d(T_{(1,2)}) \), of the ESDOF model (Figure 3b). The computed spectral
displacement is multiplied by 1.5 (modal participation factor) to obtain the wall
displacement \( \Delta_w \) (Griffith et al. 2006).
\[ \Delta_w = 1.5 S_d \left( T_{(1,2)} \right) = 1.5 \frac{S_d(T_e) \cdot g \cdot T_{(1,2)}^2}{4\pi^2} \] (10)

3) Repeat steps 1 and 2 for increasing level of IMs and develop the relationship between \( \Delta_w \) and IM. A closed form formulation of the relation between the wall peak displacement \( \Delta_w \) and the IM (i.e. PGA) is presented for the seismic demand models in Figure 3c (\( \Delta_w(1,2) = a_{(1,2)} \cdot IM \)).

2.4 Validation of the simplified method

In order to validate the proposed procedure for seismic demand modelling of out-of-plane response of URM walls, an investigation is conducted to compare the displacement predictions using the recommended equivalent period and damping ratios and the corresponding recorded displacements and damage observations from shake table test results. The results reported in the study by Meisl et al. (2007) for a three wythes load bearing masonry wall (identified in their study as wall-PD) is used for validation purposes. The wall represented a portion of the top storey of an URM school building built in early 1900s in British Columbia, Canada. Meisl's study was selected since the tested wall was subjected to increasing ground motion intensity until collapse was observed. This allows for the evaluation of the equivalent linear method at both moderate and high ground motion levels. The relevant parameters of the wall that are used as input for the simplified model are as follows: (1) the mortar compressive strength \( (f'_m) \) is equal to 6.14MPa; (2) the wall was not subjected to axial compression stress (i.e. \( \psi = 0 \)); (3) the wall height and thickness are equal to 4250mm 355mm (h/t = 12), respectively, and the wall length is equal to 1500mm; (4) the volumetric mass of the brick masonry \( (\rho) \) equals 1800kg/m\(^3\); (5) the wall was attached to a stiff braced frame that forced the top of the wall to experience the same displacements as the bottom of the wall. These boundary conditions were considered in the analysis.
conditions represent URM buildings with rigid concrete diaphragms (configuration “a” in Figure 1); (6) the shake table tests were conducted using a ground motion time-history recorded on site-class D during the 1989 Loma Prieta Earthquake in California. This record was scaled to match the uniform hazard (UHS) spectrum for Vancouver provided by the 2005 NBCC (NRCC, 2005) in the period range of 0.5s to 1.0s. The simplified analysis is conducted using the UHS for Vancouver (Figure 4a). The ESDOF capacity curve parameters for the wall was calculated based on the procedure presented in Section 2.1 (Figure 4b). The corresponding values for the limit state displacements $\Delta_1$, $\Delta_2$ and $\Delta_{\text{ins}}$ are: 10mm; 60mm and 236mm, respectively. The corresponding $T_1$ equals 0.4s and $T_2$ equals 1.0s. Figure 4c shows the predicted displacement demand using the equivalent linear procedure and the corresponding experimental displacement demand recorded from the shake table tests. The wall exhibited a stable rocking response up to PGA=1.25g and collapse occurred at dynamic excitation of PGA=1.5g. It can be observed that: (1) the application of the equivalent period ($T_1$) showed good approximation of the displacements when the displacement demands are less than 50% of the instability displacement ($\Delta < 0.5\Delta_{\text{ins}} = 118\text{mm}$); (2) on the other hand, the application of the equivalent period ($T_2$) showed a conservative but satisfactory approximation of the collapse potential of the wall (where the predicted displacement demand exceeded the $\Delta_{\text{ins}}$ (236mm) at PGA=1.5g.

3 DEVELOPMENT OF FRAGILITY FUNCTIONS

The procedure for the development of fragility functions for out-of-plane response of URM walls uses the tri-linear capacity model and the equivalent fundamental periods ($T_1$ and $T_2$) of the ESDOF as previously described in the displacement demand prediction.
model (Section 2.3). A closed form formulation of the relation between the wall peak
displacement $\Delta_w$ and the IM (i.e. PGA) is proposed as shown in Figure 3 ($\Delta_{w(1,2)} = a_{(1,2)}$).

Using the identified displacement thresholds (Table 1) and the developed seismic demand
model, the median $IM_{DS}$ that corresponds to the median threshold displacement of the
damage state can be calculated from Equation 11. The values for the standard deviation
$\beta_{DS}$ components, as expressed in Equation 2, are taken equal to the recommended values
in HAZUS Advanced Engineering Building Model (FEMA 2003) where $\beta_C = 0.25$, $\beta_D =
0.50$ and $\beta_T = 0.20$. Closed form fragility functions (Equation 1) can then be drawn using
the computed median and standard deviation for each damage state (Figure 5).

$$IM_{DS1,2} = \frac{\Delta_{DS1,2}}{a_1} \quad \text{and} \quad IM_{DS3} = \frac{\Delta_{DS3}}{a_2} \quad \text{(11)}$$

4 APPLICATION OF THE METHOD TO URM WALL BUILDINGS IN
MONTREAL

In this section, the proposed procedure for vulnerability assessment of seismic-induced
out-of-plane failure of URM walls is applied to develop fragility functions and evaluate
the potential damage, for a given seismic scenario, to an inventory of residential URM
buildings with bearing walls in Montreal. A detailed inventory of existing residential
URM buildings with bearing walls was conducted in two Montreal districts (Verdun and
Plateau Mont-Royal) (Houalard et al. 2015). The inventory analysis showed that out of
the 113 surveyed URM buildings, 74% were constructed before 1890. Figure 6a shows a
photograph for the typical URM building that was selected for further investigations as
representative of URM buildings with bearing walls. The facade walls consist of brick
masonry with 0.2m thickness. The foundations are constructed from stone masonry. The
roof system is composed of wooden floor joists bearing on the facade walls. Figure 6b shows the typical URM facade wall geometry and the assumed critical elements that are susceptible to seismic induced out-of-plane failure. The considered critical elements correspond to the idealized boundary conditions of simply supported load bearing wall with timber bearer at the top (configuration “b” in Figure 1). The geometrical and material parameters used for the simplified analysis are as follows: the average mortar compressive strength ($f'_m$) is equal to 2.0MPa; the volumetric mass of the brick masonry ($\rho$) equals 1800kg/m$^3$, the wall height is equal to 4100mm, the wall thickness is equal to 200mm ($h/t = 20$) and the wall length for W1, W2 and W3 are equal to 1350mm, 1500mm and 400mm, respectively. The corresponding values of the ($\psi$) parameters are: 1.27, 1.15 and 4.30, respectively. As previously noted, the tri-linear capacity curve is affected by the geometrical parameters of the walls. The studied walls is characterised with a high slenderness ratio ($h/t=20$) which is expected to increase the susceptibility of the walls to out-of-plane failure. Figure 7a shows the UHS corresponding to the 2010 NBCC seismic hazard for Montreal at Site-Class C which is retained for the computation of seismic demand (www.EarthquakesCanada.ca). Figure 7b presents the computed capacity curves for the three wall elements and the corresponding average capacity curve which is retained for seismic demand modelling. The corresponding values for the average limit state displacements $\Delta_1$, $\Delta_2$ and $\Delta_{ins}$ are: 5mm; 30mm and 112mm, respectively. Many idealizations of out-of-plane response have been based on the behavior of simplified unidirectional strips spanning in the vertical direction. This is mainly attributed to the fact that vertical wall segments are prone to instability effects (due to their high height to thickness ratio) whereas horizontal ones (the spandrels) are more susceptible to in-plane cracking due to the restraint effects at the spandrel ends; and
that vertical strips may be subjected to axial compressive stress due to gravity loads (for bearing walls), which affects the rocking behavior. Figure 7b shows the developed seismic demand models corresponding to the equivalent periods based on the average capacity curve \( T_1 = 0.23s \) and \( T_2 = 0.67s \). The displacement demand model corresponding to period \( T_1 \) is used for the evaluation of the median PGA of DS1 and DS2. The displacement demand model corresponding to period \( T_2 \) is used for the evaluation of the median PGA of DS3. The methodology presented in the previous section was applied to develop the corresponding out-of-plane damage fragility functions for the typical URM building facade as shown in Figure 8. The median PGA thresholds for the considered moderate (DS1), heavy (DS2) and collapse (DS3) damage states are 0.11g, 0.7g and 0.94g, respectively. The lognormal standard deviation of all damage states is 0.6. It can be observed that the median PGA thresholds for the heavy and collapse damage states have close values. This means that any slight increase in seismic PGA demand would induce dynamic instability. This is attributed to the expected response after reaching the displacement threshold for the onset of heavy damage \( \Delta_2 \); the wall response follows a strength degrading behaviour until reaching the instability displacement. Therefore, the developed fragility functions provide results that are comparable with the expected out-of-plane seismic response of URM walls.

Figure 9 shows the proportion (in %) of URM survey buildings in each damage state for seismic scenarios corresponding to 2% and 10% probability of exceedance in 50 years seismic hazard in Montreal (NRCC 2010, www.EarthquakesCanada.ca) that is considered a region of moderate seismicity. Proportion of buildings in each damage state is obtained from the difference in cumulative probability of reaching each damage state taken from Figure 8. The damage predictions show that insignificant (DS0) to moderate damage
(DS1) would be the most probable damage experienced by the considered URM buildings for the 10% in 50 years seismic hazard level (PGA = 0.12g). On the other hand, the damage predictions show that moderate damage (DS1) would be the most probable damage experienced by the considered URM buildings for the 2% in 50 years seismic hazard level (PGA = 0.32g). This indicates low risk of life-threatening injuries or casualties and low probability of debris generation. The results also indicate that low probability of out-of-plane collapse (4%) is expected for the considered scenario.

5 COMPARISON WITH EXISTING FRAGILITY FUNCTIONS

This section presents a comparative evaluation of the developed analytical fragility functions of out-of-plane failure with existing analytical and empirical fragility functions for URM buildings. Emphasis is put on out-of-plane collapse of URM walls since it is one of the main causes of casualties during earthquakes. The first comparison is conducted with the study of Sharif et al. (2007). Out-of-plane collapse fragility functions were developed using dynamic analyses of rigid body rocking model under a suite of ground motion records. The normalized fragility functions were developed in terms of the height to thickness ratio (h/t) of the URM walls and the spectral acceleration at 1.0 seconds Sa(1.0sec) as the intensity measure. Figure 10a shows the out-of-plane collapse fragility functions as lognormal functions defined by two parameters: the median value of the (h/t) ratio and the lognormal standard deviation as was originally presented in Sharif et al. (2007). The functions were originally developed for four levels of spectral accelerations at 1.0s (Sa1.0sec): 0.24g, 0.3g, 0.37g and 0.44g. The corresponding graphically interpreted median (h/t) for the four levels of Sa(1.0sec) are: 28, 26, 22 and 19, respectively. The interpreted lognormal standard deviation for all the curves was 0.26.
The Sa(1.0sec) value corresponding to the uniform hazard spectrum for 2% in 50 years in Montreal is equal to 0.14g (NRCC, 2010). The probability curve corresponding to 0.14g (shown in Figure 10 with black line) was generated based on extrapolation with a median (h/t) equals 33 and lognormal standard deviation of 0.26. The corresponding collapse probability is equal to 3% for the case study URM building facade with a ratio of height to thickness of 20 (h/t = 4.0m/0.2m). This probability is in good agreement with the 4% probability of collapse for DS3 obtained using the analytical fragility functions developed in this study.

The second comparison is conducted with the empirical collapse fragility functions for unreinforced brick masonry buildings with cement mortar class developed by Jaiswal et al. (2011) which was constructed using World Housing Encyclopaedia expert opinion survey data. Figure 10b shows the empirical collapse fragility functions as a function of the Modified-Mercalli shaking intensity (MMI). The collapse state definition for masonry buildings in their study corresponds to the failure of one or more exterior walls resulting in partial or complete failure of roof/floor. The PGA value corresponding to the uniform hazard spectrum for 2% in 50 years in Montreal is equal to 0.32g (NRCC, 2010). It was converted to (MMI=8.3) using the empirical relationship proposed by Trifunac and Brady (1975) and presented in Equation 12, where PGA is in terms of (cm/sec^2). At MMI=8.3, there would be 7% probability of collapse for the URM walls. This probability is slightly higher than the 4% probability of collapse for DS3 obtained using the analytical fragility functions developed in this study.

$$MMI = \frac{\log PGA - 0.014}{0.3} \quad (12)$$
The final comparison was conducted with observation-based fragility functions presented in Coburn and Spence (2002). These functions are based on a worldwide damage database for unreinforced brick masonry buildings. The collapse fragility function was developed in terms of the Parameterless Scale of Seismic Intensity (PSI) as shown in Figure 10c. The collapse state definition for masonry buildings in their study corresponds to the collapse of more than one wall or more than half of the roof. The PGA value corresponding to the uniform hazard spectrum for 2% in 50 years in Montreal is 0.32g. It was converted to (PSI=9) using the empirical relationship proposed by (Spence et al. 1992) as presented in Equation 13, where PGA is expressed in terms of cm/sec^2. At PSI=9, there would be 5% probability of collapse for the URM walls. This probability is in good agreement with the 4% probability of collapse for DS3 obtained using the analytical fragility functions developed in this study.

\[ \log(\text{PGA}) = 2.04 + 0.051 \times \text{PSI} \]  

(13)

The probability of collapse from Coburn and Spence (2002) fragility functions tends to get larger at higher values of PSI. For example at PSI=15, that is PGA=0.64g corresponding to an event with a longer period of return than 1/2500 years, the probability of collapse is approximately 70% compared to a probability of collapse of 26% using the analytical fragility functions developed for the URM building in Montreal (Figure 8). In terms of risk informed decision making, both collapse probabilities at that level of ground motion (PGA=0.64g) are considered high enough to tag the building as a high risk for collapse that would require detailed investigation for seismic retrofit and mitigation. The difference in the predicted collapse probability between the two sets of fragility functions is mainly attributed to the difference in the methods and assumptions.
used for the generation of these functions. The collapse fragility function developed in this study is based on an analytical procedure for specific URM building parameters (e.g. geometry and material properties) and using site-specific response spectrum for Montreal. On the other hand, the collapse fragility function developed by Coburn and Spence (2002) is based on statistical analysis of post-earthquake damage reports for thousands of URM buildings in different countries with variable geometrical and mechanical parameters. Therefore, this comparison shows the importance of the development of fragility functions that reflect the specific characteristics of the considered building and local seismic settings for reliable prediction of the seismic risks.

6 RESEARCH SIGNIFICANCE

The main contribution of this paper is the development and validation of a simplified step-by-step procedure for the generation of seismic fragility functions for out-of-plane failure of URM buildings. There is a lack of such procedures in the literature that can be used for vulnerability assessment of a portfolio of buildings especially in regions of moderate seismicity such as Eastern Canada. In the absence of post-earthquake damage observation in these regions, seismic vulnerability modelling commonly integrates existing engineering knowledge and models for capacity, demand and damage state thresholds for the development of fragility functions corresponding to seismic failure mechanisms of a specific construction system. This study integrates the existing capacity model developed by Doherty et al. (2002) for out-of-plane response of URM walls with a new simplified seismic demand model based on existing knowledge in seismic response of URM wall validated against experimental results in the literature. It also introduces displacement based damage state thresholds established from the analysis of damage
progression and observations of out-of-plane loaded URM walls as described in the available literature on experimental tests on URM walls. To the authors’ knowledge, this study presents one of the first attempts to propose and validate a simplified step-by-step procedure for the development of fragility functions of out-of-plane loaded URM walls using site-specific geometrical and material parameters. The proposed simplified procedure described in this paper defers from related studies in the literature (e.g. Sharif et al. 2007; Jaiswal et al. 2011 and Coburn and Spence 2002) as discussed in the following points. (1) Sharif et al. (2007) used extensive dynamic time history analyses with multiple earthquake records on a generic model of URM walls to generate fragility functions that depend on one parameter (h/t). On the other hand, the procedure proposed in this study considers site-specific geometrical and material parameters and applies an alternative simplified seismic demand model with less computational effort compared to dynamic time history analysis. This is particularly important for the case of regional scale vulnerability assessment of a portfolio of buildings. (2) Jaiswal et al. (2011) used expert opinion based fragility functions for URM buildings from a worldwide survey, which are mainly based on judgement rather than engineering analysis on site-specific buildings as in the proposed procedure. (3) Fragility functions developed by Coburn and Spence (2002) are based on statistical analysis of post-earthquake damage reports for thousands of URM buildings in different countries with variable geometrical and mechanical parameters.

7 CONCLUSION

This paper presented a procedure for the development of analytical fragility functions for out-of-plane failure of URM wall buildings. An important feature of the developed
fragility analysis procedure is the simplicity and reliability of its application to large number of buildings within a region with reduced computational time. Fragility functions that correlate the probability of damage to a seismic intensity measure (e.g. peak ground acceleration, PGA) were developed to evaluate the vulnerability of representative URM buildings in Montreal. The study evaluated the structural characterisation of existing URM load bearing buildings in Montreal region to identify typical facade properties that are susceptible to out-of-plane failure. A simplified probabilistic nonlinear static based procedure was developed to evaluate the seismic demand using an equivalent ESDOF model. The seismic demands were then compared to displacement capacities to develop the corresponding fragility functions for moderate, heavy and collapse damage states. The developed fragility functions were then used to evaluate the out-of-plane seismic vulnerability for URM buildings corresponding to the design level seismic hazard ground motion with 10% and 2% probability of exceedance in 50 years as defined in the National Building Code of Canada (NRCC 2010). The damage predictions show that moderate damage would be the most probable damage to be experienced by the considered URM buildings. This indicates low risk of life-threatening injuries or casualties and low probability of debris generation. On the other hand, the results indicate that low probability of out-of-plane collapse (4%) is expected for the considered scenario. The predicted collapse probability using the developed fragility functions showed good agreement with the corresponding probabilities estimated using existing analytical, expert-opinion and observation based collapse fragility functions for URM buildings. It should be noted that the inventory of buildings in Montreal showed also significant number of wood buildings with brick veneer cladding which are susceptible to out of plane damage. Future development in the procedure should include the out of plane
response of brick veneers with modifications to consider the interaction with the wood backing system.
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<tr>
<th>Damage state</th>
<th>Displacement threshold criteria</th>
<th>Post-earthquake condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Insignificant(DS0)</td>
<td>Elastic response. Displacement demand $\leq \Delta_1$</td>
<td>Immediate occupancy. Restoration not required for structural performance.</td>
</tr>
<tr>
<td>Moderate (DS1)</td>
<td>Rocking response without strength degradation $\Delta_1 &lt; \text{Displacement demand} \leq \Delta_2$</td>
<td>Limited safety. Low risk of life-threatening injury. Repairable damage. Repoint spalled mortar for restoration.</td>
</tr>
<tr>
<td>Heavy/Extreme (DS2)</td>
<td>Rocking response with strength degradation $\Delta_2 &lt; \text{Displacement demand} \leq \Delta_{\text{ins}}$</td>
<td>Near collapse. The risk of life-threatening injury is significant. Wall replacement is required.</td>
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<tr>
<th>Support type</th>
<th>Representation</th>
<th>Equivalent parapet model</th>
</tr>
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<tr>
<td><strong>a)</strong></td>
<td>Rigid load bearing simply-supported wall with top and base reactions at the leeward face</td>
<td>[ h_{\text{equiv}} = \frac{h}{4(1 + \psi)} ]</td>
</tr>
<tr>
<td></td>
<td>Slab boundary condition</td>
<td>[ t_{\text{equiv}} = t ]</td>
</tr>
<tr>
<td><strong>b)</strong></td>
<td>Rigid load bearing simply-supported wall with top at center line and base reaction at the leeward face</td>
<td>[ h_{\text{equiv}} = \frac{(1 + 0.75\psi)(1 + \psi)}{4(1 - \psi)^2} ]</td>
</tr>
<tr>
<td></td>
<td>Timber bearer boundary condition</td>
<td>[ t_{\text{equiv}} = t \frac{(1 + 0.75\psi)}{1 + \psi} ]</td>
</tr>
</tbody>
</table>

\( \psi \) : ratio of overburden

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