Canadian Journal of Civil Engineering
Revue canadienne de génie civil

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

Journal:	<i>Canadian Journal of Civil Engineering</i>
Manuscript ID	cjce-2016-0555.R2
Manuscript Type:	Article
Date Submitted by the Author:	21-Jul-2017
Complete List of Authors:	Abo El Ezz, Ahmad; Natural Resources Canada, Geological Survey of Canada Houalard, Clémentine; Léon Grosse Nollet, Marie-José; Ecole de technologie supérieure, Genie de la construction; Assi, Rola; ETS, Génie de la construction
Is the invited manuscript for consideration in a Special Issue? :	N/A
Keyword:	Seismic vulnerability assessment, fragility analysis, unreinforced masonry, out-of-plane damage

SCHOLARONE™
Manuscripts

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

Ahmad Abo-El-Ezz, Ph.D.
Research Scientist
Geological Survey of Canada, Natural Resources Canada
490, rue de la Couronne, Québec
Canada, G1K 9A9

Clémentine Houalard,
Engineer, Léon Grosse, 21 avenue Salvador Allende, 69500 Bron, France.

Marie-José Nollet, ing.,Ph.D.
Professor
Département de génie de la construction,
École de Technologie Supérieure, Université du Québec
1100 Notre-Dame Ouest,
Montréal, QC
Canada, H3C 1K3

Rola Assi, ing.,Ph.D.
Assistant Professor
Département de génie de la construction,
École de Technologie Supérieure, Université du Québec
1100 Notre-Dame Ouest,
Montréal, QC
Canada, H3C 1K3

Corresponding Author:

Ahmad Abo-El-Ezz, Ph.D.
Research Scientist
Geological Survey of Canada, Natural Resources Canada
490, rue de la Couronne, Québec
Canada, G1K 9A9
Phone: +1(514) 572-7217
E-mail: ahmad.aboelezz@canada.ca

Number of words: 7 670 text + 1 table + 10 figures = 10 420 words

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 (DS_i), given the IM, is defined in Equations 1 and 2 (Kircher et al. 1997).

$$P[DS | IM] = \Phi \left[\frac{1}{\beta_{DS}} \ln \left(\frac{IM}{IM_{DS}} \right) \right] \quad (1)$$

$$\beta_{DS} = \sqrt{\beta_C^2 + \beta_D^2 + \beta_T^2} \quad (2)$$

Where IM_{DS} is median value of the IM at which the building reaches the threshold of damage state DS, and Φ is standard normal cumulative distribution function. β_{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 β_D , the variability in the seismic capacity corresponding to damage state β_C and the variability in the threshold of the damage state β_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 (ψ) 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

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

202 The observed response of cracked out-of-plane wall subjected to out-of-plane loading is
203 curvilinear (Doherty et al. (2002)). It can be predicted through the equivalent tri-linear
204 capacity model and the classical rigid body bilinear equilibrium model shown in Figure 2.
205 Four parameters are used to draw the tri-linear capacity model: the wall instability
206 displacement Δ_{ins} (Equation 3), the empirical displacement values Δ_1 (Equation 4), and Δ_2
207 (Equation 5), and the maximum force F_i (Equation 6). Equations 3 to 8 are expressed in
208 terms of the equivalent thickness t_{equiv} and equivalent height h_{equiv} for the equivalent
209 parapet model shown in Figure 1. Δ_1 is defined as an empirical displacement at which
210 wall's force-displacement relation reaches its maximum strength (F_{max} in Figure 2); Δ_2 is
211 defined as an empirical displacement at which wall's maximum force plateau intersects
212 with the rigid body bilinear model; Δ_{ins} is defined as the failure displacement at which the
213 wall becomes unstable and F_i (Equation 6) is defined as the actual wall lateral strength
214 which is calculated as a function of the rigid body lateral strength F_o (Equation 7). The
215 effective mass was considered as equal to (3/4) of the mass of the wall (M) for the
216 computation of the lateral strength. The reader is referred to (Doherty et al. (2002);
217 Derakhshan et al. (2013) for detailed derivation of the listed equations. Experimental
218 studies were conducted to define the empirical displacements Δ_1 and Δ_2 as a function of
219 Δ_{ins} (Doherty et al. 2002; Griffith et al. 2004; Derakhshan et al. 2013). Derakhshan et al.
220 (2013) observed that the wall instability displacement Δ_{ins} and displacement values Δ_1 ,
221 and Δ_2 are sensitive to the crack height ratio (β), the overburden ratio (ψ) and the mortar
222 compressive strength (f'_j). The crack height ratio (β) 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, β 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_{emp} (Percentage of Maximum rigid Resistance) (Equation 8). The PMR_{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_{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_i) 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_{emp} .

$$\Delta_{ins} = \frac{2}{3} t_{equiv} \quad (3)$$

$$\Delta_1 = 0.04 \Delta_{ins} \quad (4)$$

$$\Delta_2 = (1 - 0.009 PMR_{emp}) \Delta_{ins} \quad (5)$$

$$F_i = 0.9 (PMR_{emp} \cdot F_0) \quad (6)$$

$$F_0 = \frac{M_e \cdot g \cdot t_{equiv}}{h_{equiv}} = \frac{3}{4} \frac{M \cdot g}{h_{equiv}} \left(\frac{t_{equiv}}{h_{equiv}} \right) \quad (7)$$

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

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, Δ_{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

260 damage that has potential for human injury. Four damage states are identified in the
261 FEMA-306 report (FEMA 1998) for rigid-body rocking motion of URM walls including:
262 insignificant, moderate, heavy and extreme damage. The insignificant damage state
263 represents hairline cracks at floor/roof lines and mid-height of stories. The moderate
264 damage represents cracks at floor/roof lines and mid-height of stories with mortar
265 spalling to full depth of joint and possibly out-of-plane offsets along cracks. The heavy
266 damage state represents spalling of units along crack plane with out-of-plane offsets
267 along cracks and significant crushing/spalling of bricks at crack locations. The extreme
268 damage state represents a wall with threatened vertical-load-carrying ability, significant
269 out-of-plane movement at top and bottom of the wall and significant crushing/spalling of
270 bricks at crack locations. These damage states are only described in terms of qualitative
271 characterisation without identifying associated displacement thresholds. The FEMA-306
272 report (FEMA 1998) stated that “*as rocking increases, the mortar and masonry units at*
273 *the crack locations can be degraded, and residual offsets can occur at the crack planes.*
274 *The ultimate limit state is that the walls rock too far and overturn*”. Lumantarana et al.
275 (2006) considered three displacement thresholds for minor, moderate and collapse
276 damage states. The minor damage threshold at which the wall is expected to undergo first
277 cracking was associated with a wall displacement of 5mm. The displacement limit at
278 moderate damage was arbitrarily defined as equivalent to half of the wall thickness. URM
279 walls subject to displacement exceeding this limit are expected to have a fully developed
280 crack pattern that forms a collapse mechanism. The displacement limit at collapse was
281 defined at the wall thickness. Based on the interpretation of the above references and the
282 authors engineering judgment, four damage states with three corresponding displacement
283 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 Δ_l 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 Δ_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 Δ_{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 (ζ) 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 (T_2) (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.

$$T_{1,2} = 2\pi \sqrt{\frac{M_e}{K_{(1,2)}}} = 2\pi \sqrt{\frac{0.75M}{K_{(1,2)}}} = 2\pi \sqrt{\frac{0.75 \times \rho_m \cdot h \cdot t \cdot L}{F_i / \Delta_{1,2}}} \quad (9)$$

In this study, two seismic demand models with two corresponding displacement thresholds were developed. The first model applies the equivalent period (T_1) for comparison with the displacement thresholds that are less than 50% of Δ_{ins} , denoted Δ_1 for the moderate damage. The second model applies the equivalent period (T_2) for comparison with the displacement thresholds that are greater than 50% of Δ_{ins} (i.e. the displacement threshold for the onset of collapse Δ_{ins}), denoted Δ_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 (T_1 and T_2) 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 (Δ_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 Δ_w (Griffith et al. 2006).

$$\Delta_w = 1.5 S_d(T_{(1,2)}) = 1.5 \frac{S_a(T_e) \cdot g \cdot T_{(1,2)}^2}{4\pi^2} \quad (10)$$

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

331 2.4 Validation of the simplified method

332 In order to validate the proposed procedure for seismic demand modelling of out-of-plane
 333 response of URM walls, an investigation is conducted to compare the displacement
 334 predictions using the recommended equivalent period and damping ratios and the
 335 corresponding recorded displacements and damage observations from shake table test
 336 results. The results reported in the study by Meisl et al. (2007) for a three wythes load
 337 bearing masonry wall (identified in their study as wall-PD) is used for validation
 338 purposes. The wall represented a portion of the top storey of an URM school building
 339 built in early 1900s in British Columbia, Canada. Meisl's study was selected since the
 340 tested wall was subjected to increasing ground motion intensity until collapse was
 341 observed. This allows for the evaluation of the equivalent linear method at both moderate
 342 and high ground motion levels. The relevant parameters of the wall that are used as input
 343 for the simplified model are as follows: (1) the mortar compressive strength (f'_j) is equal
 344 to 6.14MPa; (2) the wall was not subjected to axial compression stress (i.e. $\psi = 0$); (3) the
 345 wall height and thickness are equal to 4250mm 355mm ($h/t = 12$), respectively, and the
 346 wall length is equal to 1500mm; (4) the volumetric mass of the brick masonry (ρ) equals
 347 1800kg/m³; (5) the wall was attached to a stiff braced frame that forced the top of the
 348 wall to experience the same displacements as the bottom of the wall. These boundary

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 Δ_1 , Δ_2 and Δ_{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 ($\Delta < 0.5\Delta_{ins} = 118mm$); (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 Δ_{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 Δ_w and the IM (i.e. PGA) is proposed as shown in Figure 3 ($\Delta_{w(1,2)} = a_{(1,2)} \cdot IM$).

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 β_{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).

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

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

394 roof system is composed of wooden floor joists bearing on the facade walls. Figure 6b
395 shows the typical URM facade wall geometry and the assumed critical elements that are
396 susceptible to seismic induced out-of-plane failure. The considered critical elements
397 correspond to the idealized boundary conditions of simply supported load bearing wall
398 with timber bearer at the top (configuration “b” in Figure 1). The geometrical and
399 material parameters used for the simplified analysis are as follows: the average mortar
400 compressive strength (f_j) is equal to 2.0MPa; the volumetric mass of the brick masonry
401 (ρ) equals 1800kg/m³, the wall height is equal to 4100mm, the wall thickness is equal to
402 200mm ($h/t = 20$) and the wall length for W1, W2 and W3 are equal to 1350mm,
403 1500mm and 400mm, respectively. The corresponding values of the (ψ) parameters are:
404 1.27, 1.15 and 4.30, respectively. As previously noted, the tri-linear capacity curve is
405 affected by the geometrical parameters of the walls. The studied walls is characterised
406 with a high slenderness ratio ($h/t=20$) which is expected to increase the susceptibility of
407 the walls to out-of-plane failure. Figure 7a shows the UHS corresponding to the 2010
408 NBCC seismic hazard for Montreal at Site-Class C which is retained for the computation
409 of seismic demand (www.EarthquakesCanada.ca). Figure 7b presents the computed
410 capacity curves for the three wall elements and the corresponding average capacity curve
411 which is retained for seismic demand modelling. The corresponding values for the
412 average limit state displacements Δ_1 , Δ_2 and Δ_{ins} are: 5mm; 30mm and 112mm,
413 respectively. Many idealizations of out-of-plane response have been based on the
414 behavior of simplified unidirectional strips spanning in the vertical direction. This is
415 mainly attributed to the fact that vertical wall segments are prone to instability effects
416 (due to their high height to thickness ratio) whereas horizontal ones (the spandrels) are
417 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 ($T_1 = 0.23\text{s}$) and ($T_2 = 0.67\text{s}$). 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 (Δ_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 ($Sa(1.0sec)$): 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.

465 The $S_a(1.0\text{sec})$ value corresponding to the uniform hazard spectrum for 2% in 50 years in
 466 Montreal is equal to 0.14g (NRCC, 2010). The probability curve corresponding to 0.14g
 467 (shown in Figure 10 with black line) was generated based on extrapolation with a median
 468 (h/t) equals 33 and lognormal standard deviation of 0.26. The corresponding collapse
 469 probability is equal to 3% for the case study URM building facade with a ratio of height
 470 to thickness of 20 ($h/t = 4.0\text{m}/0.2\text{m}$). This probability is in good agreement with the 4%
 471 probability of collapse for DS3 obtained using the analytical fragility functions developed
 472 in this study.

473 The second comparison is conducted with the empirical collapse fragility functions for
 474 unreinforced brick masonry buildings with cement mortar class developed by Jaiswal et
 475 al. (2011) which was constructed using World Housing Encyclopaedia expert opinion
 476 survey data. Figure 10b shows the empirical collapse fragility functions as a function of
 477 the Modified-Mercalli shaking intensity (MMI). The collapse state definition for masonry
 478 buildings in their study corresponds to the failure of one or more exterior walls resulting
 479 in partial or complete failure of roof/floor. The PGA value corresponding to the uniform
 480 hazard spectrum for 2% in 50 years in Montreal is equal to 0.32g (NRCC, 2010). It was
 481 converted to ($\text{MMI}=8.3$) using the empirical relationship proposed by Trifunac and Brady
 482 (1975) and presented in Equation 12, where PGA is in terms of (cm/sec^2). At $\text{MMI}=8.3$,
 483 there would be 7% probability of collapse for the URM walls. This probability is slightly
 484 higher than the 4% probability of collapse for DS3 obtained using the analytical fragility
 485 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.

$$\text{LogPGA} = 2.04 + 0.051 \text{ PSI} \quad (13)$$

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 $\text{PGA}=0.64\text{g}$ 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 ($\text{PGA}=0.64\text{g}$) 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

554 fragility analysis procedure is the simplicity and reliability of its application to large
555 number of buildings within a region with reduced computational time. Fragility functions
556 that correlate the probability of damage to a seismic intensity measure (e.g. peak ground
557 acceleration, PGA) were developed to evaluate the vulnerability of representative URM
558 buildings in Montreal. The study evaluated the structural characterisation of existing
559 URM load bearing buildings in Montreal region to identify typical facade properties that
560 are susceptible to out-of-plane failure. A simplified probabilistic nonlinear static based
561 procedure was developed to evaluate the seismic demand using an equivalent ESDOF
562 model. The seismic demands were then compared to displacement capacities to develop
563 the corresponding fragility functions for moderate, heavy and collapse damage states.
564 The developed fragility functions were then used to evaluate the out-of-plane seismic
565 vulnerability for URM buildings corresponding to the design level seismic hazard ground
566 motion with 10% and 2% probability of exceedance in 50 years as defined in the National
567 Building Code of Canada (NRCC 2010). The damage predictions show that moderate
568 damage would be the most probable damage to be experienced by the considered URM
569 buildings. This indicates low risk of life-threatening injuries or casualties and low
570 probability of debris generation. On the other hand, the results indicate that low
571 probability of out-of-plane collapse (4%) is expected for the considered scenario. The
572 predicted collapse probability using the developed fragility functions showed good
573 agreement with the corresponding probabilities estimated using existing analytical,
574 expert-opinion and observation based collapse fragility functions for URM buildings. It
575 should be noted that the inventory of buildings in Montreal showed also significant
576 number of wood buildings with brick veneer cladding which are susceptible to out of
577 plane damage. Future development in the procedure should include the out of plane

578 response of brick veneers with modifications to consider the interaction with the wood
579 backing system.

580

Draft

581 REFERENCES

- 582 Abo-El-Ezz A, Lefebvre K and Nollet M-J 2014. "Seismic performance assessment of
583 masonry infill reinforced concrete buildings in Eastern Canada". The IES Journal Part
584 A: Civil & Structural Engineering, 7(3): 207-218.
- 585 Abo-El-Ezz A, Nollet M-J and Nastev M 2013. "Seismic fragility assessment of low-rise
586 stone masonry buildings". Earthquake Engineering and Engineering Vibration, 12(1):
587 87-97.
- 588 Abo-El-Ezz A, Nollet M-J and Nastev M 2015. "Assessment of earthquake-induced
589 damage in Quebec city, Canada". International Journal of Disaster Risk Reduction,
590 12: 16-24.
- 591 Abrams DP, AlShawa O, Lourenço PB and Sorrentino L 2017. "Out-of-Plane Seismic
592 Response of Unreinforced Masonry Walls: Conceptual Discussion, Research Needs,
593 and Modeling Issues." International Journal of Architectural Heritage, 11(1): 22-30.
- 594 Antunez G, Abo-El-Ezz A, Nollet M-J and Khaled A 2015. "Analyse de la vulnérabilité
595 sismique hors-plan des bâtiments de maçonnerie de pierre de l'Est canadien:
596 Application aux bâtis du Vieux-Québec et du Vieux-Montréal". Canadian Journal of
597 Civil Engineering, 42(12): 1125-1134.
- 598 ATC 2005. Field manual: post-earthquake safety evaluation of buildings, ATC-20-1,
599 Second Edition, Applied Technology Council, Redwood City, California.
- 600 Borzi B, Crowley H and Pinho R 2008. "Simplified pushover-based earthquake loss
601 assessment (SP-BELA) method for masonry buildings". International Journal of
602 Architectural Heritage, 2(4): 353-376.
- 603 Coburn A and Spence R 2002. Earthquake protection, 2nd edition, J. Wiley, Chichester,
604 England.
- 605 D'Ayala D, Speranza E 2003 "Definition of collapse mechanisms and seismic
606 vulnerability of masonry structures." Earthquake Spectra 19(3): 479–509.
- 607 D'Ayala D, Meslem A, Vamvatsikos D, Porter K and Rossetto T 2015. Guidelines for
608 analytical vulnerability assessment of low/mid-rise buildings, vulnerability global
609 component project. GEM Technical Report, GEM Foundation, Pavia, Italy.
- 610 De Felice G and Giannini R 2001. "Out-of-plane seismic resistance of masonry walls."
611 Journal of Earthquake Engineering, 5(2): 253–271
- 612 Derakhshan H, Griffith MC and Ingham JM 2013. "Out-of-plane behavior of one-way
613 spanning unreinforced masonry walls". Journal of Engineering Mechanics, 139(4): 409-
614 417.

- 615 Derakhshan H, Dizhur DY, Griffith MC and Ingham JM 2014. "Seismic assessment of
616 out-of-plane loaded unreinforced masonry walls in multi-storey buildings." Bulletin of
617 the New Zealand Society for Earthquake Engineering, 47(2): 119-138.
- 618 Doherty K, Griffith MC, Lam N and Wilson J 2002. "Displacement based seismic
619 analysis for out-of-plane bending of unreinforced masonry walls". Earthquake
620 Engineering and Structural Dynamics, 31(4): 833-850.
- 621 FEMA 1998. FEMA-306, Evaluation of Earthquake Damaged Concrete and Masonry
622 Wall Buildings - Basic Procedures Manual, prepared by the Applied Technology
623 Council (ATC-43), Redwood City, California.
- 624 FEMA 2003. HAZUS-MH-MR5, Earthquake Loss Estimation, Advanced Engineering
625 Building Module, Technical and user's manual. Washington, D.C.
- 626 FEMA P58-1 2012. Seismic performance assessment of buildings (volume 1-
627 Methodology). Federal Emergency Management Agency, Washington.
- 628 Griffith MC, Lam N and Wilson J 2006. "Displacement-based assessment of the seismic
629 capacity of unreinforced masonry walls in bending". Australian Journal of Structural
630 Engineering, 6(2): 119-132.
- 631 Griffith MC, Lam N, Wilson J and Doherty K 2004. "Experimental investigation of
632 unreinforced brick masonry walls in flexure". Journal of Structural Engineering,
633 130(3): 423-432.
- 634 Griffith MC, Magenes G, Melis G and Picchi L 2003. "Evaluation of out-of-plane
635 stability of unreinforced masonry walls subjected to seismic excitation". Journal of
636 Earthquake Engineering, 7(01): 141-169.
- 637 Houalard C, Abo-El-Ezz A and Nollet M-J 2015 "Displacement Response Analysis of
638 Out-of-Plane Loaded URM walls: Comparison with Shake Table Tests". Proceedings
639 of the 11th Canadian Conference on Earthquake Engineering, Victoria, BC, Canada,
640 Paper no. 94204.
- 641 Ingham JM and Griffith MC 2010. "Performance of unreinforced masonry buildings
642 during the 2010 Darfield (Christchurch, NZ) earthquake". Australian Journal of
643 Structural Engineering, 11(3): 207-224.
- 644 Jaiswal K, Wald D and D'Ayala D 2011. "Developing empirical collapse fragility
645 functions for global building types". Earthquake Spectra, 27(3): 775-795.
- 646 Jeong SH and Elnashai AS 2007. "Probabilistic Fragility Analysis Parameterized by
647 Fundamental Response Quantities". Engineering Structures, 29(6): 1238-1251.
- 648 Kircher CA, Nassar AA, Kustu O and Holmes WT 1997. "Development of building
649 damage functions for earthquake loss estimation". Earthquake Spectra, 13(4): 663-682.

- 650 Krawinkler H, Osteraas JD, McDonald BM and Hunt JP 2012. "Development of
651 damaged fragility functions for URM chimneys and parapets". Proceedings of the 15th
652 World Conference in Earthquake Engineering, Lisbon, Portugal, Paper no. 4622.
- 653 Lagomarsino, S. and Resemini, S. 2009. "The Assessment of Damage Limitation State in
654 the Seismic Analysis of Monumental Buildings", *Earthquake Spectra*, 25(2): 323-346.
- 655 Lumantarna E, Vaculik J, Griffith MC, Lam N and Wilson J 2006. "Seismic fragility
656 curves for un-reinforced masonry walls". Proceedings of the Australian Earthquake
657 Engineering Society Conference, Canberra, Australia: pp.33-40.
- 658 Magenes, G and Penna, A 2011. "Seismic design and assessment of masonry buildings in
659 Europe: recent research and code development issues." Proceedings of the 9th
660 Australasian masonry conference, Queenstown, New Zealand,.
- 661 Meisl CS, Elwood KJ and Ventura CE 2007. "Shake table tests on the out-of-plane
662 response of unreinforced masonry walls". *Canadian Journal of Civil Engineering*,
663 34(11): 1381-1392.
- 664 Nollet M-J, Abo-El-Ezz A and Hébert M 2016. "Experimental evaluation of mechanical
665 properties of replica materials for seismic assessment of traditional masonry buildings
666 in Eastern Canada". *NED University Journal of Research - Structural Mechanics*, 13(1):
667 1-14.
- 668 NRCC 2005. National Building Code of Canada, Associate Committee on the National
669 Building Code, National Research Council of Canada, Ottawa, Ontario, Canada.
- 670 NRCC 2010. National Building Code of Canada, Associate Committee on the National
671 Building Code, National Research Council of Canada, Ottawa, Ontario, Canada.
- 672 Porter K, Farokhnia K, Vamvatsikos D. and Cho IH 2015. "Guidelines for vulnerability
673 assessment of buildings and nonstructural elements". GEM Technical Report, GEM
674 Foundation, Pavia, Italy.
- 675 Restrepo-Velez LF and Magenes G 2004. "Simplified procedure for the seismic risk
676 assessment of unreinforced masonry buildings". Proceedings of the 13th World
677 Conference on Earthquake Engineering, Vancouver, Canada, Paper no.2561.
- 678 Ruiz-García J and Negrete M 2009. "Drift-based fragility assessment of confined
679 masonry walls in seismic regions". *Engineering Structures*, 31(1): 125-137.
- 680 Sharif I, Meisl CS and Elwood KJ 2007. "Assessment of ASCE 41 height-to-thickness
681 ratio limits for URM walls". *Earthquake Spectra*, 23(4): 893-908.
- 682 Spence R, Coburn A, Pomonis A and Sakai S 1992. "Correlation of ground motion with
683 building damage: the definition of a new damage-based seismic intensity scale".
684 Proceedings of the 10th World Conference on Earthquake Engineering, Madrid.

- 685 Sorrentino L, Kunnath S, Monti G and Scalora, G. 2008. "Seismically Induced One-
686 Sided Rocking Response of Unreinforced Masonry Façades", Engineering Structures,
687 30(8): 2140-2153.
- 688 Taylor RL 2000. FEAP, A Finite Element Analysis Program, Department of Civil and
689 Environmental Engineering, University of California at Berkeley.
- 690 Trifunac MD and Brady AG 1975. "On the correlation of seismic intensity scales with
691 the peaks of recorded strong ground motion". Bulletin of the Seismological Society of
692 America, 65(1): 139-162.
- 693

Draft

694 **LIST OF TABLES**

695 Table 1: Proposed damage states and corresponding displacement threshold criteria for
696 out-of-plane response of URM wall.

697

Draft

698 Table 1: Proposed damage states and corresponding displacement threshold criteria for
 699 out-of-plane response of URM wall.

Damage state	Displacement threshold criteria	Post-earthquake condition
Insignificant(DS0)	Elastic response. Displacement demand $\leq \Delta_1$	Immediate occupancy. Restoration not required for structural performance.
Moderate (DS1)	Rocking response without strength degradation $\Delta_1 < \text{Displacement demand} \leq \Delta_2$	Limited safety. Low risk of life-threatening injury. Repairable damage. Repoint spalled mortar for restoration.
Heavy/Extreme (DS2)	Rocking response with strength degradation. $\Delta_2 < \text{Displacement demand} \leq \Delta_{\text{ins}}$	Near collapse. The risk of life-threatening injury is significant. Wall replacement is required.
Collapse (DS3)	Wall overturning. Displacement demand $> \Delta_{\text{ins}}$	Collapsed wall. High risk of life-threatening injury.

700
 701
 702

LIST OF FIGURES

- Figure 1: Configurations of URM walls and equivalent parapet model as recommended by Doherty et al. (2002).
- Figure 2: Representation of the tri-linear capacity model as recommended by Doherty et al. (2002).
- Figure 3: Schematic for the development of the seismic displacement demand model: (a) Definition of equivalent periods and associated displacements, (b) Seismic displacement associated to a given PGA and (c) Seismic demand models.
- Figure 4: (a) UHS corresponding to the 2005 NBCC seismic hazard at Vancouver; (b) Computed ESDOF capacity curve for the URM wall tested by Meisl et al. (2007) and (c) Experimental and analytical displacement demands for the tested URM wall.
- Figure 5: Illustration of the development of damage state medians from the seismic demand model and the corresponding fragility functions.
- Figure 6: (a) A photograph for a typical URM building with load bearing walls and (b) Average geometrical parameters of typical URM facade wall and the assumed critical elements (dimensions are in meters).
- Figure 7: (a) UHS corresponding to the 2010 NBCC hazard for Montreal at Site Class C; (b) Capacity curves for the considered wall elements for the URM facade wall and (c) Seismic demand models for the out-of-plane response of the case study URM facade wall.
- Figure 8: Out-of-plane damage fragility functions for the case study URM facade wall.
- Figure 9: Proportion of URM buildings in each damage state corresponding to ground motion at Montreal with probability of exceedance of 10% ($PGA=0.12g$) and 2% ($PGA=0.32g$) in 50 years, respectively.
- Figure 10: (a) Out-of-Plane collapse fragility function (Sharif et al. 2007); probability of collapse for $S_a(1.0sec)=0.14g$ and $(h/t)=20$ is indicated by the arrow, (b) collapse fragility function for unreinforced brick masonry construction (Jaiswal et al. 2011); probability of collapse for $MMI=8.3$ is indicated by the arrow, (c) collapse fragility functions for unreinforced brick masonry buildings (Spence et al. 1992); probability of collapse for $PSI=9$ is indicated by the arrow.

Support type	Representation	Equivalent parapet model
a) Rigid load bearing simply-supported wall with top and base reactions at the leeward face		$h_{equiv} = \frac{h}{4(1 + \psi)}$ $t_{equiv} = t$
b) Rigid load bearing simply-supported wall with top at center line and base reaction at the leeward face		$h_{equiv} = h \frac{(1 + 0,75\psi)}{4(1 + \psi)^2}$ $t_{equiv} = t \frac{(1 + 0,75\psi)}{1 + \psi}$
ψ : ratio of overburden		

Figure 1: Configurations of URM walls and equivalent parapet model as recommended by Doherty et al. (2002).

Figure 2: Representation of the tri-linear capacity model as recommended by Doherty et al. (2002).

Draft

742

743

744 Figure 3: Schematic for the development of the seismic displacement demand model: (a)
 745 Definition of equivalent periods and associated displacements, (b) Seismic displacement
 746 associated to a given PGA and (c) Seismic demand models.

747

748
749

750
751
752
753
754

Figure 4: (a) UHS corresponding to the 2005 NBCC seismic hazard at Vancouver; (b) Computed ESDOF capacity curve for the URM wall tested by Meisl et al. (2007) and (c) Experimental and analytical displacement demands for the tested URM wall.

755

756

757 Figure 5: Illustration of the development of damage state medians from the seismic
 758 demand model and the corresponding fragility functions.

759

Figure 6: (a) A photograph for a typical URM building with load bearing walls and (b) Average geometrical parameters of typical URM facade wall and the assumed critical elements (dimensions are in meters).

Figure 7: (a) UHS corresponding to the 2010 NBCC hazard for Montreal at Site Class C; (b) Capacity curves for the considered wall elements for the URM facade wall and (c) Seismic demand models for the out-of-plane response of the case study URM facade wall.

Figure 8: Out-of-plane damage fragility functions for the case study URM facade wall.

Figure 9: Proportion of URM buildings in each damage state corresponding to ground motion at Montreal with probability of exceedance of 10% (PGA=0.12g) and 2% (PGA=0.32g) in 50 years, respectively.

Figure 10: (a) Out-of-Plane collapse fragility function (Sharif et al. 2007); probability of collapse for $S_a(1.0\text{sec})=0.14\text{g}$ and $(h/t)=20$ is indicated by the arrow, (b) collapse fragility function for unreinforced brick masonry construction (Jaiswal et al. 2011); probability of collapse for $MMI=8.3$ is indicated by the arrow, (c) collapse fragility functions for unreinforced brick masonry buildings (Spence et al. 1992); probability of collapse for $PSI=9$ is indicated by the arrow.