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

IMPACT OF STONE MASONRY MECHANICAL PROPERTIES ON SEISMIC VULNERABILITY ASSESSMENT OF UNREINFORCED MASONRY BUILDINGS

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ABSTRACT

In Eastern Canada, seismic vulnerability analysis of unreinforced stone masonry buildings relies on analytical methods consisting of structural modeling and evaluation of the likelihood for a given building to experience damage from earthquake of a given intensity. In this paper, the main components of a vulnerability assessment procedure are reviewed with emphasis on the significance of masonry mechanical properties on damage estimates. An experimental program is presented which was developed to assess mechanical properties of typical stone masonry assemblies composed of lime-stone blocks joined with cement/lime mortar commonly used in heritage buildings construction in Eastern Canada. The experimental joint shear bond, compressive and diagonal shear strength parameters were used to develop seismic vulnerability functions expressed as function of the mean damage factor (MDF) corresponding to the expected repair cost ratio for increasing seismic intensity measure (IM=Sa0.3sec). The influence of the mechanical properties on damage assessment is evaluated. The results provided a quantitative assessment of the impacts of mechanical properties on the predicted seismic induced repair costs for stone masonry buildings. This has a direct impact on the decisions of risk assessment studies for seismic mitigation and retrofit that are related to the expected repair costs for the corresponding site-specific seismic hazard intensity.

KEYWORDS: *masonry, diagonal shear strength, stone, brick, damage, seismic risk*

INTRODUCTION

Severe earthquakes are typically irregular events and those which lead to catastrophic consequences are relatively rare. However, if not adequately addressed, these rare but disastrous events could lead to significant loss of life and property while more frequent moderate

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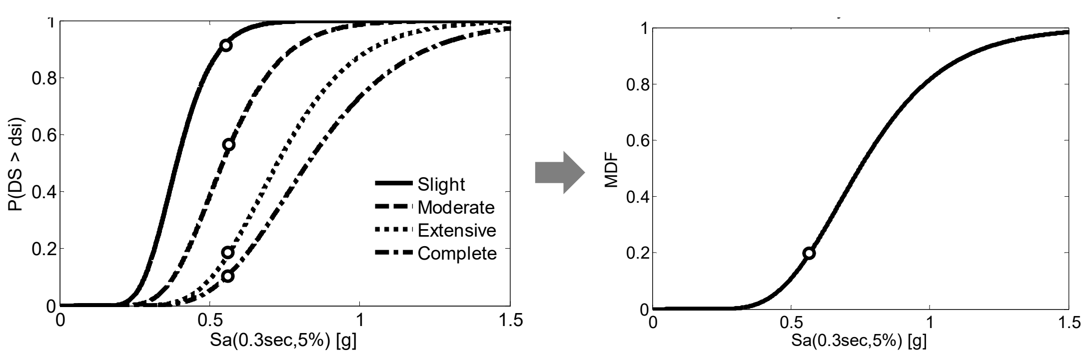
earthquakes could cause severe damage to the most vulnerable infrastructures and buildings such as unreinforced masonry structures (URM). Seismic risk assessment is a well-recognized approach to improve mitigation, preparedness and emergency response measures. It gives a measure of the negative impacts of eventual earthquakes: structural damage, disruption of economic activity, social losses, etc., and their likelihood. It involves the definition of: (1) seismic hazard, i.e, the measure of the probability of occurrence of ground shaking of a given intensity for a geographic area and time period, (2) exposure, i.e., population and the built environment at risk, and (3) vulnerability which introduces the concept of susceptibility to damage, loss and injuries [1].

Existing vulnerability analysis methods rely on damage data derived from post-earthquake surveys, expert opinion, analytical simulations of structural models, or combinations of these respectively [1][2]. Figure 1 shows an illustration of seismic fragility functions and the corresponding vulnerability function in terms of an intensity measure IM=Sa(0.3s). The fragility function represents the probability of damage and the vulnerability function represents the mean damage factor (MDF) which is defined as the repair to replacement cost ratio of the building. The successive steps of the analytical vulnerability analysis consists of: (1) definition of the seismic motion in terms of a intensity measure (IM), e.g., spectral acceleration at a particular period; (2) structural analysis using capacity curves to estimate the seismic displacement demand [3]; (3) damage analysis by comparing the predicted displacement demand to damage state thresholds ; and (4) loss analysis using damage factors (DF_{dsi}) corresponding to the repair cost ratio for different damage states probabilities ($P[DS_i|Sa(0.3s]$ (Equation 1). The results are summed to give a MDF which when multiplied by the value of the building provides an estimate of economic losses [4].

$$MDF = \sum_{dsi=1}^{4} P[DS_i \mid S_a(0.3\text{s})].DF_{dsi} \qquad (1)$$



(a) (b)

Figure 1: Vulnerability analysis: (a) Seismic fragility functions and, (b) Vulnerability function. The dots illustrate the integration of the individual damage states from the fragility functions to obtain the respective vulnerability function

In this study, the DFs were calibrated based on the observed post-earthquake damage data [5]: 2% for slight damage, 10% for moderate damage, 50% for extensive damage, and 100% for complete damage. The four steps are repeated for increasing IM to develop the vulnerability function of the building (Figure 1b).

The development of capacity and fragility functions for URM buildings often rely on simple mechanical models using geometric and mechanical properties input parameters. In Eastern Canada, however, little is known about the characteristics of load bearing walls in historical stone masonry structures, such as wall composition, geometry and materials [6][7]. There is also limited reported information regarding the mechanical properties of those URM walls. Data from literature are then often used for seismic risk studies, leading to difficulty in providing a reliable prediction of the seismic resistance and performance of URM buildings. The sensitivity of seismic damage and loss estimates to the variation of input parameters and their relative importance has been the subject of numerous past studies [8][9][10][11].

The objective of this paper is to illustrate how mechanical properties of unreinforced stone masonry buildings can impact building damage estimates and economic loss. The main steps of a vulnerability assessment procedure for URM buildings are reviewed and the main input parameters are identified. An experimental program is then presented which was developed to assess mechanical properties of typical stone masonry assemblies composed of lime-stone blocks joined with cement/lime mortar commonly used in heritage buildings construction in Eastern Canada. The impact of the mechanical properties on the yield strength of the building capacity curve is assessed and the sensitivity of the resulting MDF is discussed.

VULNERABILITY ASSESSMENT OF LOW-RISE STONE MASONRY BUILDINGS

Analytical vulnerability assessment of buildings requires first the definition of a prototype buildings representative of the inventory. The focus is put here on low-rise stone masonry buildings in the Old Québec City and Old Montréal representing an immeasurable architectural and cultural heritage and a significant portion of pre-code construction in these geographical areas [12][13]. Built to resist gravity loads only, these buildings generally offer poor resistance to lateral seismic loads. Damage to these type of stone masonry buildings from past earthquakes around the world can be attributed to: (i) inadequate structural integrity due to the lack of connection between stone masonry structural walls and wooden floors and roofs, and (ii) inadequate structural resistance which results in typical shear cracking and disintegration of stone walls and their partial or total collapse. The high seismic risk related to stone masonry buildings is even more aggravated due to their location in densely populated urban centers so that the consequences of failure of these structures tend to be severe with regards to human casualties, heritage damage and economic losses [15].

Stone masonry building prototype

The building prototype is shown in Figure 2. It represents typical 2-story buildings constructed during the 18th and mid-19th century. The massive façade walls are relatively thick, ranging from 0.4 to 0.6 m, and have regular window and door openings on the front and rear sides of the building. The typical story height ranges from 2.7 to 3.3 m. Lateral fire walls are of the same thickness as the façade walls. The massive façade and lateral fire walls provide the vertical and lateral resistance. The stone walls were built of limestone blocks bonded with lime mortar. The openings are bridged with wooden lintels. The typical floor is made of wood joists resting on the façade walls. This prototype model was used for the displacement-based damage model as input to develop the fragility functions and for the development of the capacity curves.

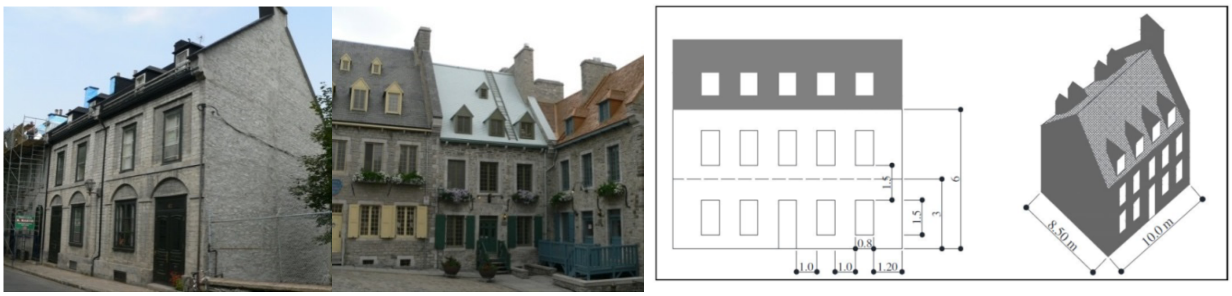


Figure 2: Typology of low-rise stone masonry building and prototype

Displacement-based damage model

The structural seismic performance of unreinforced masonry buildings was assessed using mechanical quantities, such as inter-story drift, which is related to the physical condition of the building following an earthquake. These mechanical quantities should not exceed given threshold values defined as damage states (e.g. slight, moderate, extensive and complete). Simplified displacement-based procedures can be applied assuming that damage is related to the capacity of in-plane loaded walls to sustain the top of the wall horizontal displacement (drift) [15][16][17]. Note that masonry walls can be subjected to an out-of-plane failure mode orthogonal to the earthquake direction, depending mainly on the quality of floor to wall connection. In this case, walls can fail locally, especially at top stories where dynamic amplification is higher. However, for walls that are properly anchored to floors, the out-of-plane behavior should not be critical and the building damage is governed by the in-plane behavior. This assumption is valid for most of the existing stone masonry buildings in the Old Québec City and Old Montréal, which were provided with steel anchors and other improvements in the floor to wall connections.

The masonry structure was modelled as an equivalent single degree of freedom system (ESDOF) with effective global parameters. The displacement threshold relative to a given structural damage state at the effective height of the ESDOF, equivalent to the spectral displacement ($S_{d,DSi}$) in Equation 2, is obtained based on the assumed deformed shape of the structure as follows:

$$S_{d,Dsi} = \theta_{DS1} k_1 H + (\theta_{DSi} - \theta_{DS1}) k_2 h_s \qquad (2)$$

where, H is total height of the building, h_s is height of the first story, θ_{DS1} is drift threshold for the first story walls at the elastic limit, θ_{DSi} is the drift threshold for the first story walls at higher damage states in the non-linear domain, k_1 is the effective height coefficient that converts a multiple degree of freedom (MDOF) into an ESDOF system, and k_2 is the effective height of the first story walls in the inelastic range when openings are present. The respective values for the studied 2-story buildings are k_1 = 0.72 and k_2 = 0.95 (see [16] for details of the computation of these two coefficients).

Drift thresholds for masonry walls are generally identified from laboratory experiments on masonry wall elements under static cyclic loading [14][17][18][19]. They are identified on the drift-shear force envelope curve for the stone masonry walls shown in Figure 3b, where they correspond to: flexural cracking (θ_{DS1}), shear cracking (θ_{DS2}), maximum shear strength (θ_{DS3}), and ultimate deformation at 20% loss of strength (θ_{DS4}) [17][20]. They are considered respective damage thresholds for the slight, moderate, extensive, and complete damage states.

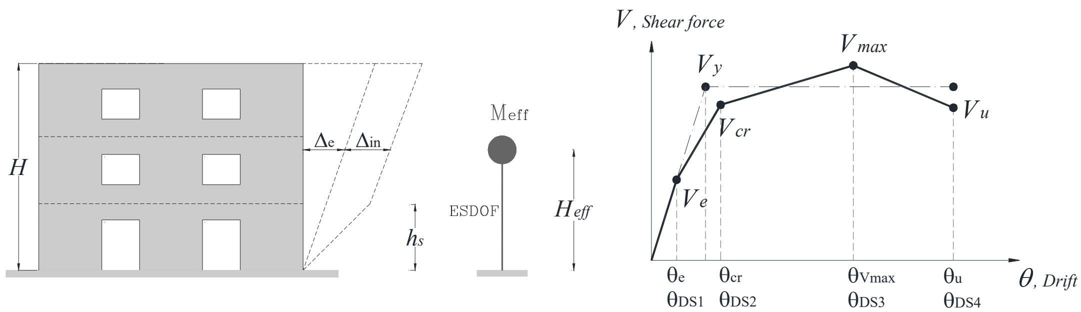


Figure 3: Simplified model for computation of damage states

In the absence of experimental values for masonry walls with similar mechanical material properties, drift thresholds could be derived from representative literature experimental data [21]. However, a large variety in specimen-to-specimen mechanical properties, geometry and stone block arrangement of the tested walls contribute to increase the dispersion in the identification of higher damage state thresholds.

Capacity evaluation from a simplified mechanical model

In order to develop a representative capacity model for the considered building typology, a simplified mechanical model was used to develop the approximate elastic-plastic base shear - roof displacement relationships. Previous research studies [14][22], have shown that the simplified elastic-plastic models provided good approximation of the global system capacity when compared with experimental results on several scaled models of masonry buildings. Such simplification reduces the computation time and more importantly idealizes the system with less number of parameters which is highly desirable when conducting regional scale vulnerability modelling. The elastic deformation of the building is approximated by a linear function up to the point where the shear capacity of the wall is attained. The inelastic deformation is assumed as perfectly plastic and concentrated at the first story only, which is the typical damage observed in masonry buildings [14]. The effective stiffness of the elastic part is determined using the secant stiffness at the capacity, V_y, and is selected such that the area under the bilinear curve is equivalent to the area under the experimental curve (Figure 3) [14]. The base shear strength (yield capacity) of the building in a given direction is assumed equal to the sum of the shear strengths of the first story walls in that direction (Figure 4).

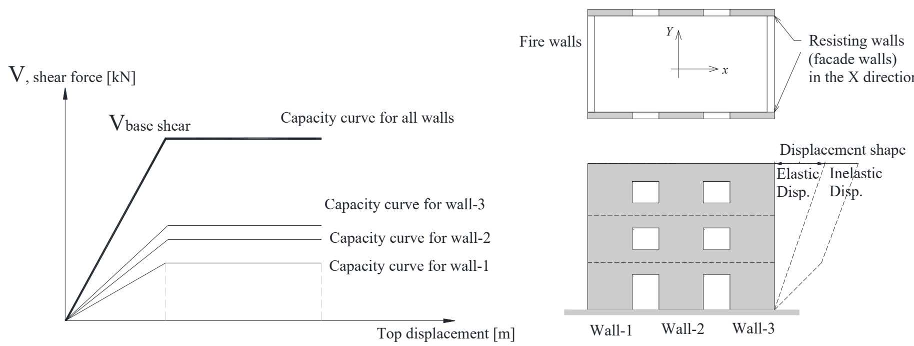


Figure 4: Mechanical model for capacity curve evaluation of stone masonry buildings

To combine the damage model and capacity model with the seismic hazard, the base shear – roof displacement relationship shown in Figure 4 is converted to capacity curves for an equivalent single degree of freedom system in terms of spectral acceleration-spectral displacement relationship. This conversion has the advantage to allow a direct comparison with the seismic demand represented with response spectra.

Two mechanical criteria were considered to evaluate the shear strength of the first story walls based on the expected failure mechanism given by Equation 3: the flexural strength corresponding to reaching of the flexural rocking failure criterion, V_r, and the shear strength at the attainment of diagonal tension cracking failure criterion, V_{ds}. These two criteria provided good approximation of the shear strength of stone masonry walls when compared with experimental results [16][17][18][19][20].

$$V_r = \left(\frac{L^2.t.\sigma_0}{h_p}\right).\left(1-\frac{\sigma_0}{0.85 f_m}\right)$$
$$V_{ds} = \left(\frac{L.t.f_{dt}}{b}\right).\sqrt{\left(1+\frac{\sigma_0}{f_{dt}}\right)} \tag{3}$$

where, L is the wall length; t is the wall thickness; h_p is the height of first floor pier, σ_0 is the average axial stress; f_m is the compressive strength of masonry; f_{dt} is the diagonal shear strength of masonry; and b is a factor depending on the aspect ratio (height/length= h_p/L), b=1.0 for $h_p/L<1$, b=1.5 for $h_p/L>1.5$ or b= h_p/L for $1<h_p/L<1.5$. The governing failure mechanism depends on the axial stress, aspect ratio, compression and diagonal shear strength of masonry.

In the first studies conducted on unreinforced stone masonry buildings presented in Figure 2, no site-specific information on the physical mechanical properties was available. Studies were conducted using values from the literature for masonry mechanical properties [13][21], which may introduce significant variability in the developed capacity models.

EXPERIMENTAL ASSESSMENT OF STONE MASONRY MATERIAL PARAMETERS

An experimental program was developed to reduce the uncertainties in the evaluation of the capacity of stone URM walls and improve damage estimates for seismic risk studies. This program aimed to assess the mechanical parameters for stone masonry assemblies composed of lime-stone blocks joined with cement/lime mortar commonly used in heritage buildings construction in Eastern Canada, including: compressive strength of lime mortar and limestone blocks, compressive strength of the stone masonry assembly, joint shear bond strength and diagonal shear strength parameters, as well as drift-shear force envelope under cyclic loading.

Characterization of the stone masonry walls

A typical URM wall cross-section considered as potentially vulnerable to earthquake loading was identified from a review of documentation on series of rehabilitation and conservation projects of heritage stone masonry buildings [23]. Figure 5 shows the elevation and cross-section of a representative three-leaf stone masonry wall typically composed of limestone blocks joined with hydraulic lime and cement mortar. Test specimens were constructed with dimensions of the stone representing average values from the investigated conservation projects.

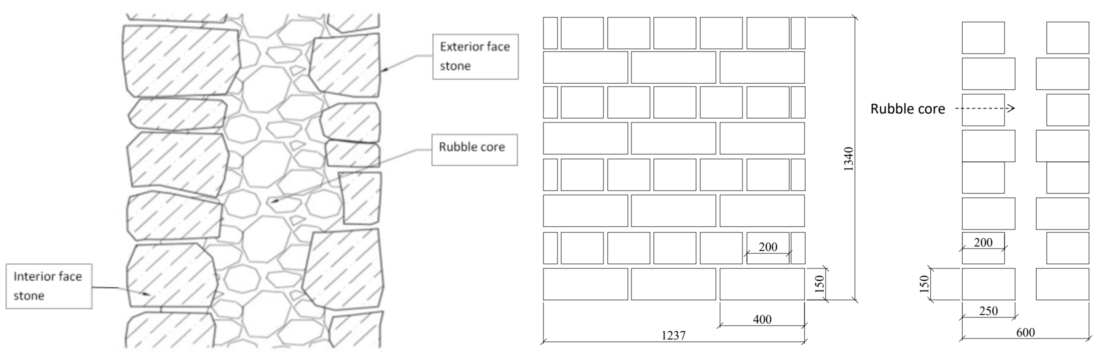


Figure 5: URM wall cross-section and experimental test specimen

Experimental program and results

The experimental program included three distinct phases as shown on Figure 6.

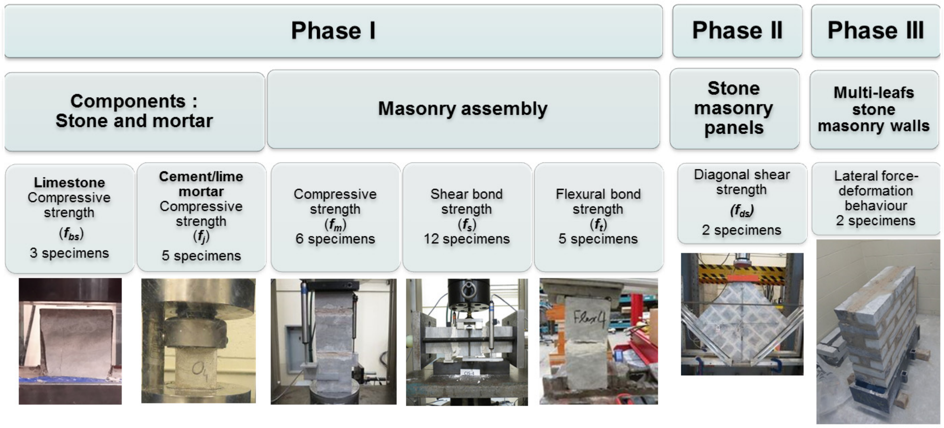


Figure 6: Experimental program

The first phase consisted of characterizing the mechanical properties of the stones and the mortar, as well as the compressive and joint shear sliding strength and flexural bond strength, of the stone-mortar assembly. The second phase consisted in evaluating the diagonal shear strength of stone masonry panels. Mechanical properties from those two first phases are summarized in Table 1 for the masonry assembly. They were used to define the capacity model of the building prototype as described previously. The third phase consisted in evaluating the lateral force-deformation behavior of the representative wall specimens in Figure 5 to update the drift threshold values for the stone URM prototype building.

Table 1: Summary of the results of Phase I and II of the experimental program

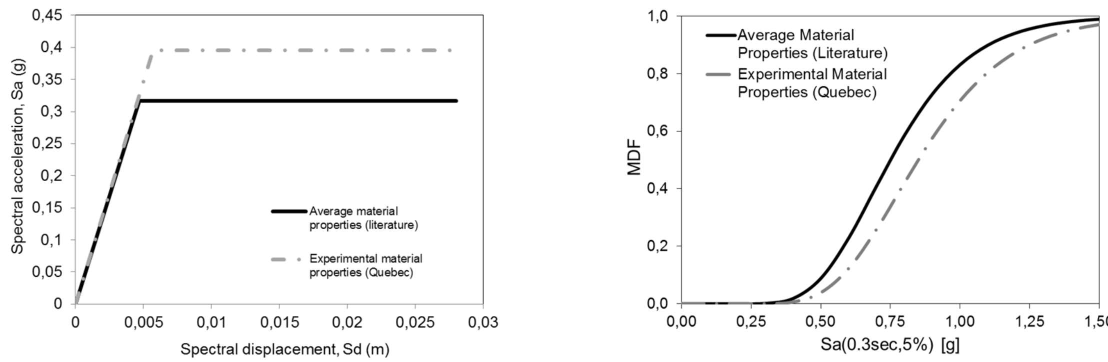
Phase I					
Compressive strength (6 specimens)		**Shear bond strength (12 specimens)**		**Flexural bond strength (5 specimens)**	
f'_m (MPa)	33.2 ± 3.2	Cohesion, C (MPa)	0.56	f_t (MPa)	0.23 ± 3.2
E_m (MPa)	2823 ± 186	Coefficient of friction μ	0.85		
Phase II – Diagonal shear strength(2 panels)					
f_{ds} (MPa)	0.37 ± 0.05				
G (MPa)	487.17 ± 9.60				

IMPACT OF MECHANICAL PROPERTIES ON ECONOMIC LOSS

In the absence of regional specific mechanical properties, risk modelling is usually carried out using literature data for a given construction material. To illustrate the impact of using regional specific mechanical properties on economic losses for risk studies, a sensitivity study is conducted.

Figure 7a shows the capacity curves developed using Equation 3 with two sets of material mechanical properties: (i) average mechanical properties collected from the literature [11][21] representing mainly South European constructions and, (ii) the material properties from the experimental program (Table 1) compatible with Quebec stone masonry constructions.

It can be observed that the yield strength (Sa_y) of the equivalent bilinear model (Figure 3b) derived from experimental mechanical properties equals 0.4g, which is larger than the yield strength from the literature (Sa_y=0.32g) by 25%. To quantify the impact of such a variation in the capacity curve yield strength on the expected economic losses, vulnerability functions were developed for both cases. Following the vulnerability analysis steps presented in the introduction, capacity curves in Figure 7a were combined with the fragility function for the low-rise stone masonry building illustrated in Figure 1a. To develop fragility functions, the displacement-based damage model uses mechanical properties of the masonry and drift thresholds generally identified from experimental cyclic loading tests. As these test results are not yet available, literature values for drift threshold [21] are used to derive the median displacement and standard deviation for the fragility function, however increasing the dispersion in the identification of higher damage state thresholds [11].



(a) (b)

Figure 7: Vulnerability functions: (a) Capacity curves, (b) Variation in yield spectral acceleration (Sa_y).

Figure 7b gives the resulting vulnerability functions for the Québec stone masonry prototype building using experimental data and using average literature material properties. These functions give the mean damage factor (MDF), as defined by Equation 1, in terms of the intensity measure IM Sa(0.3sec). The deviation in the MDF due to an increase of +25% in the (Sa_y) from the experimental capacity curve for three different levels of ground motion intensity IM, results in a reduction in the MDF of 51%, 42% and 34% corresponding to (Sa0.3s) = 0.4g, 0.5g, and 0.6g, respectively. The sensitivity of MDF to the variation of the mechanical parameters is mainly related to their direct effects on the yield strength of the building (Equation 3). With increased yield strength, the displacement demands would be reduced. The reduction in displacement demand would reduce the probability of damage and consequently would lead to a reduction in the expected repair cost ratio.

CONCLUSION

This paper summarized the analytical procedure for the development of seismic hazard compatible vulnerability functions for existing stone masonry buildings. The capacity of URM buildings is defined from mechanical properties of the masonry. Using literature data instead of site specific mechanical properties could lead to significant variability in the capacity models.

An experimental program was carried out to assess the mechanical parameters for stone masonry assemblies composed of lime-stone blocks joined with cement/lime mortar which were commonly used in heritage buildings construction in Eastern Canada. The experimental joint shear bond, compressive and diagonal shear strength parameters were used to define the capacity in terms of yield strength and the resulting seismic vulnerability expressed as function of the mean damage factor (MDF) vs. a structure-independent IM (Sa0.3sec). The capacity curve yield acceleration increased by +25% compared to capacity curve obtained from average mechanical parameters from the literature. The increase in the (Sa_y) results in a reduction in the MDF of 51%, 42% and 34% for three different levels of ground motion intensity (Sa0.3s) = 0.4g, 0.5g, and 0.6g, respectively. The results provided a quantitative assessment of the effects of material strength parameters on the predicted seismic induced repair costs for stone masonry buildings. This has a direct impact on the decision process in risk assessment studies for seismic mitigation and retrofit that are related to the expected damage and repair costs for the corresponding site-specific seismic hazard intensity. The MDF could still be sensitive to the median damage state threshold values. The current research activities by the authors are focused on updating the drift threshold for stone masonry buildings based on compatible geometrical and material properties and evaluate the sensitivity of the vulnerability functions to the updated thresholds.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the financial support provided by the Natural Sciences and Engineering Research Council of Canada and the École de technologie supérieure in Montreal. The authors also wish to thank the firm EVOQ Architects and Maçonnerie L.M.R. Inc. for their contribution to the project.

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