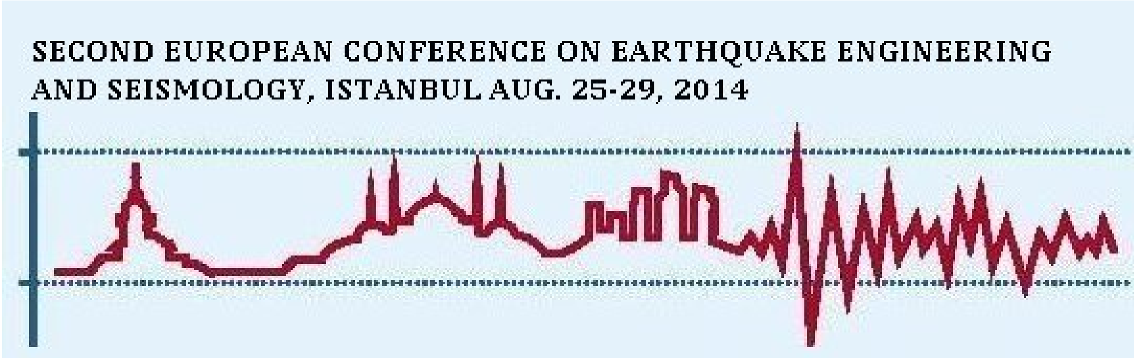

REGIONAL SEISMIC RISK ASSESSMENT OF MASONRY BUILDINGS

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ABSTRACT

A framework for quantitative assessment of regional seismic risk of unreinforced masonry buildings is presented. It considers region-specific inventory of buildings, definition of the seismic hazard and evaluation of respective vulnerabilities. Structural vulnerability is derived from the fragility functions relating the intensity of the seismic motion to the expected damage. The old historic center of Quebec City, Canada, a UNESCO heritage city, was selected as a study area due to the concentration of masonry buildings and their high heritage value. An inventory of the structural characteristics of these buildings was conducted and the respective capacity and fragility functions were developed and validated against empirical fragility functions based on damage data observed during past earthquakes. The proposed framework was then applied to assess the seismic vulnerability of the old historic center. Results for a scenario event of magnitude 6.2 at hypocentral distance 15 km indicate that approximately 39% of the stone masonry buildings and 33% of the brick masonry buildings would suffer various levels of damage.

INTRODUCTION

The seismic hazard in Eastern Canada is generally defined as low to moderate with the exception of the high seismicity Charlevoix region. Although large urban centers such as Quebec City are faced with relatively low earthquake hazard, the seismic risk increases in older neighborhoods characterized by important concentration of buildings constructed prior to the introduction of seismic design provisions. Unreinforced masonry buildings represent a significant portion among those pre-code constructions (Abo-El-Ezz et al., 2011). Such is the case with the numerous historic buildings in the Old Quebec City made of stone masonry with immeasurable architectural and cultural heritage. Built to resist gravity loads only, these buildings generally offer poor resistance to lateral seismic loads. Past strong earthquakes have occasionally caused damages to unreinforced masonry buildings in Quebec. Occurred damage to stone masonry buildings is usually attributed to inadequate structural integrity due to the lack of connection between stone masonry structural walls and wooden floors and roofs, and inadequate structural resistance which results in typical shear cracking and disintegration of stone walls and their partial or total collapse (Tomaževič and Lutman, 2007). The relatively high seismic risk akin to stone masonry buildings is aggravated due to their location in densely populated town centers in a way that the consequences of failure of these structures tend to be even more severe in terms of social and economic losses.

Strong earthquakes have occurred in the past and may occur again. Therefore, the first necessary step in developing seismic retrofitting strategy and preparation of pre-disaster mitigation plans consists of the assessment of negative impacts to the existing building stock including the unreinforced masonry buildings. The present study is a part of the ongoing collaborative research conducted jointly by the École de technologie supérieure Montréal and Natural Resources Canada – Geological Survey of Canada on the development and implementation of standardized tools for quantitative assessment of earthquake risk at regional scale. In this paper, a framework is proposed for quantitative assessment of earthquake risk of unreinforced masonry buildings

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which can also be applied to any other structural type. It considers region-specific inventory of the existing building stock, definition of the seismic hazard, and evaluation of respective vulnerabilities. Structural vulnerability represents the key component of the framework. It is derived from fragility functions relating the intensity of the seismic motion to the expected damage (Coburn and Spence, 2002). The old historic center of Quebec City, Canada, was selected as a study area due to the increased concentration of stone masonry buildings and their high heritage value. An inventory and structural characterization of these buildings is presented first. The collected structural properties are used to develop respective fragility functions defined by the expected degree of damage that could result under different levels of seismic loading. The proposed framework was then applied to quantify the potential damage of the 1,220 existing buildings in the Old Quebec City for a scenario event with magnitude 6.2 and hypocentral distance 15 km (M6.2D15). The results indicate that approximately 39% of the stone masonry buildings and 33% of the brick masonry buildings would suffer various levels of damage.

RISK ASSESSMENT FRAMEWORK

The proposed quantitative seismic risk assessment framework consists of three successive models (Figure 1): (1) building inventory model based on classification according to construction material, structural system, height and design level; (2) probabilistic or deterministic seismic hazard model, using a ground-motion prediction equation compatible to the seismo-tectonic settings and soil conditions in the study region which estimates the potential ground shaking intensity in terms of a structure-independent intensity measure, IM (e.g. spectral acceleration at a particular period); and (3) vulnerability model represented with seismic hazard compatible fragility functions in terms of structure-independent IM defined by the spectral acceleration Sa(0.3sec). The damage estimates are given in terms of the number of damaged buildings, magnitude of damage and their distribution.

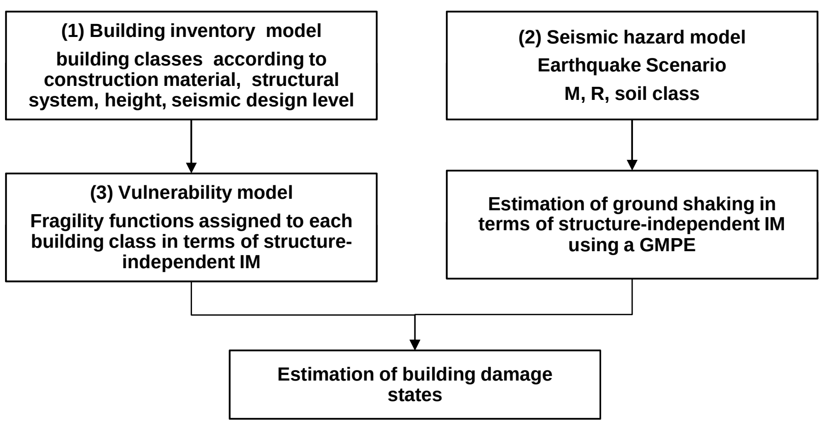

Figure 1: Framework for quantitative seismic risk assessment.

INVENTORY

A detailed inventory was carried out first to characterize the existing historic stone masonry buildings in the Old Quebec City. Besides the field survey conducted in summer 2011, the inventory consisted of comprehensive review of available architectural reports, theses, and historic documents (Vallières, A. 1999). The configuration and construction methods of the stone masonry buildings have evolved gradually since the beginning of the colony in the early 17th century. Still, among the typologies reported by Vallières, three dominant types of stone masonry buildings have been selected as structural prototypes for further analyses (Figure 2). These building types were built mainly during the 18th until the mid-19th century. The masonry was made of local limestone or sandstone with lime mortar. The façade walls are relatively thick, ranging from 0.4 to 0.6 m, sometimes up to 1.5m at the base, and have regular narrow window and door openings on both sides of the building. They are typically one to three storeys with storey height ranging from 2.75m to 3.35m. Lateral fire walls are also built with the same thickness as the façade walls. The typical floor is made of wood resting on the façade walls with a light roof frame. The resistance of these buildings to eventual lateral loads is provided by the thick perimeter walls (façade and fire walls) in both directions.

Past strong earthquakes in the province of Quebec have occasionally caused damages to unreinforced masonry buildings, e.g., 1663 Charlevoix earthquake (M7+), 1732 Montreal earthquake (M5.8), or the more recent 1925 Charlevoix earthquake (M6.2). Analysis of the occurred damage show damages mainly to unreinforced masonry walls and chimneys. Due to the frequent presence of religious structures, the reported seismic damage is often related to churches, chapels and other sacral buildings. These are predominantly stone masonry constructions. The observed damage consists mainly of out of plane failure of facades, side walls or gables, and damage to bell towers.

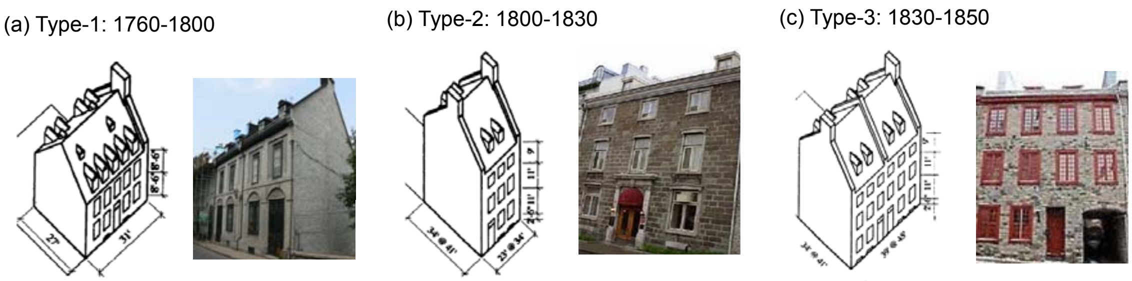

Figure 2: Typical stone masonry buildings present in the Old Quebec City according to year of construction: a) 1760-1800, b) 1800-1830, and c) 1830-1850.

VULNERABILITY MODELING PROCEDURE

The vulnerability of a typical building type or a class of buildings can be assessed applying different methods such as observed damage from past major earthquakes for which adequate records of the seismic motion are available (empirical method), expert opinion, analytical methods involving simplified mathematical models of structural response of a building or a type of buildings, time-domain numerical modeling of structural response, and by a combination of any of these methods. In the absence of observed earthquake damage patterns or insufficient data, as is the case in Eastern Canada, analytical methods are preferred, often as the only feasible solution to the problem.

Vulnerability assessment based on analytical modeling relies on a suite of procedures with capacity curves and fragility functions as essential input components. Capacity curves describe the nonlinear structural behavior and are generally obtained from pushover analysis as a relationship between roof displacement and lateral load capacity (FEMA 356, 2000). On the other hand, fragility function defines the probability of being in or exceedeing a given physical damage state, e.g., slight, moderate, extensive or complete damage state (Coburn and Spence, 2002). Fragility functions are usually given as lognormal distribution functions of the considered seismic IM, typically defined with a spectral acceleration for given period (Sa) and damping. They can also be conditioned on a structure specific IM, e.g., inelastic spectral displacement. In this case they are defined as displacement based fragility functions.

The conducted vulnerability modeling was inspired by the procedure employed in Hazus, the well-known loss estimation methodology developed by the US Federal Emergency Management Agency – FEMA (FEMA, 2012). The capacity curves and the displacement based fragility functions for stone masonry buildings were determined by Abo-El-Ezz et al. (2013). The vulnerability modeling procedure is graphically presented in Figure 3.

For a given building type, the vulnerability modeling starts with the development of the seismic scenario. Structure-independent IMs, Sa0.3s and Sa1.0s fully define the simplified shape input response spectrum using a suitable ground motion prediction equation. The structural analysis is conducted in the spectral acceleration vs. spectral displacement domain (Sa-Sd domain). The structural response is evaluated using the modified capacity spectrum method (CSM) (ATC-40, 1996). In the standard CSM, the performance point is obtained assuming that the nonlinear response of the system is modeled as a linear equivalent single degree of freedom system with increased period and effective damping, both related to the ductility demand (displacement demand over the yield displacement). To avoid the computationally costly iterative procedure for the structural displacement response, i.e., the performance point, the CSM procedure was amended according to the suggestions proposed by Porter (2009). The amended CSM shown in Figure 3a, determines the performance point for the considered earthquake magnitude-distance scenario on the capacity curve in the Sa-Sd domain. The corresponding effective damping is calculated from the ductility-damping relationships, as shown in Figure 3a. The associated values of the structure-independent IMs of the site-specific response spectrum for a given soil class (Sa0.3s for 5% damping), are obtained next using the spectral reduction factor relationship between the performance point Sa with effective damping and the Sa0.3s with 5% damping.

The second step proceeds forward from the performance point into the set of previously developed displacement based fragility functions to determine the probability of damage states (Figure 3b). The obtained probabilities are then ranked with respect to the computed IM (indicated with hollow dots in Figure 3c). To establish a complete set of fragility functions in terms of the structure-independent IMs, the procedure is repeated for gradually increasing shaking intensities, i.e., increasing demand response spectra. The computed probabilistic damage states are arranged in tabular format for the respective structure-independent IM and a lognormal cumulative probability functions is fitted with proper mean and standard deviation to provide suitable hazard compatible seismic fragility functions. In addition to the vulnerability assessment of individual building or a portfolio of buildings, the above procedure revealed as a powerful tool for conducting rapid damage assessment at a regional scale.

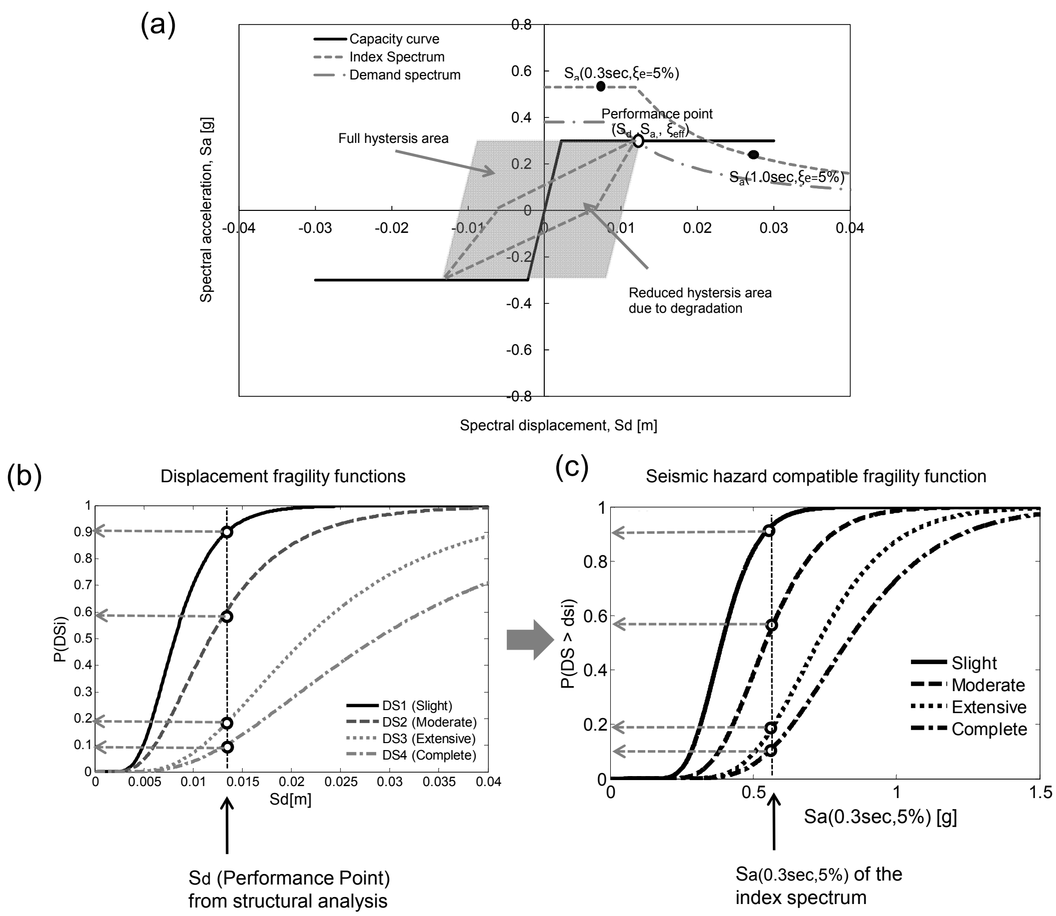

Figure 3.Illustration of the vulnerability modeling procedure: (a) definition of the performance point; (b) estimation of the probability of damage states; and (c) conversion of the hazard compatible fragility functions.

COMPARISON WITH OBSERVATION BASED FRAGILITY FUNCTIONS

To validate the presented procedure, the developed fragility functions for stone masonry buildings are compared with reported observation-based fragility functions from a worldwide damage database for different typologies of masonry buildings including adobe, stone and unreinforced and reinforced brick masonry (Spence et al., 1992; Coburn and Spence, 2002). Based on a statistical analysis conducted on damage data observed in the vicinity of recording instruments, it was concluded that the mean response spectral acceleration over a range of 0.1 to 0.3 seconds (MRSA) provides a good predictor for the damageability of masonry buildings with one to three stories. Figure 4 shows the corresponding fragility functions for stone masonry buildings. The adopted damage scale is the EMS98 (Grünthal 1998). For a comparative evaluation, the heavy damage state is compared to the extensive damage, whereas the very heavy and destruction damage state is compared to the complete damage state (Hill and Rossetto, 2008). The comparison between damage estimation using the analytical fragility functions developed in this study and the corresponding empirical functions for stone masonry buildings for three levels of the IM Sa(0.3sec)=0.4g, 0.5g, and 0.6g is shown in Figure 5.

It can be observed in Figure 5 that the analytical fragility functions present higher probabilities of no or slight damage as opposed to higher probabilities of extensive and complete damage in case of the empirical estimates. This results in an overall underestimation of the predicted damage. As well, it can be observed that the relative discrepancy in damage estimations reduces for increased IM levels to the increased probability of reaching the extensive to complete damage states. There are numerous limitations, however, when the fragility functions developed by different models are directly compared because of the different assumptions, information (structure, material properties, geometry, soil conditions) and tools used in the development process. Hence, the objective of the comparison is given rather to show the importance of the development of fragility functions that reflect the specific characteristics of the considered structures and local seismo-tectonic settings.

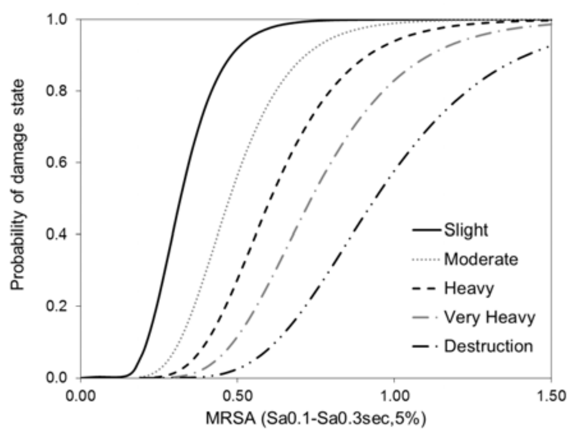

Figure 4.Observation-based fragility functions for stone masonry building class for mean response spectral acceleration – MRSA (Spence et al. 1992).

CASE STUDY

The above procedure was applied to conduct damage assessment of the existing unreinforced masonry buildings in the Old Quebec City. The assessment was performed for a hypothetical scenario M6.2R15 event which corresponds roughly to a probability of exceedence of 2% in 50 years according to the National Building Code of Canada (NBCC, 2010). The response spectrum for the selected scenario was developed using the AB06 ground motion prediction equation (Atkinson and Boore, 2006). The ground motion parameters retained for the damage assessment were the spectral accelerations Sa0.3s

(0.38g) and Sa1.0s (0.07g) as representative IMs for short and long periods and for site class B (rock), the predominant soil type in the study area.

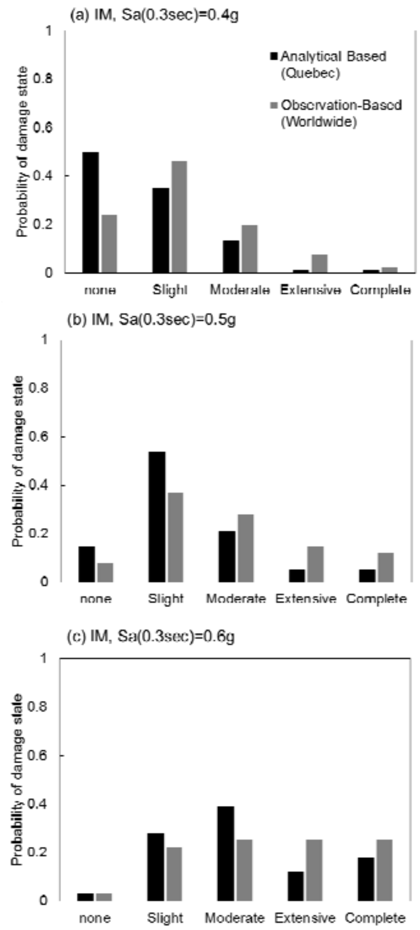

Figure 5.Comparison of damage estimations using the analytical-based fragility functions for typical stone masonry buildings in Quebec City and the observation-based fragility functions from worldwide data.

The building inventory was compiled by a combination of data from the municipal database and from a field survey. Structural data were collected for a total of 1220 buildings in the Old Quebec City as shown in Figure 6. The inventoried buildings were classified according to (1) construction material: wood, steel, concrete, masonry; (2) structural system: frame or bearing wall structure; (3) seismic design code level: pre-code for buildings designed without consideration of any specific aseismic measures (structures built before 1970s), and mid-code for buildings designed according to the existing seismic provisions (between 1970 and 1990); and (4) height: low-rise with 1 to 3 stories, mid-rise with 4 to 7 stories (Nollet et al. 2012). This classification scheme corresponds to the one employed in Hazus. A summary chart of the inventoried buildings according to the construction material type is shown in Figure 6. It can be seen that the dominant building types are the pre-code unreinforced brick and stone masonry buildings 76% or (933 buildings). 91% of the existing buildings were built before 1970 and are assumed of pre-code design level. The first seismic design provisions were introduced with the 1941 National Building Code edition, however, most of the buildings in Eastern Canada built prior to 1970 can still considered as pre-code buildings, especially those of unreinforced masonry.

Figure 7 shows the developed hazard compatible fragility functions for stone and brick masonry buildings in terms of Sa0.3sec the representative IM for buildings with short fundamental vibration period. Steeper fragility functions in the case of stone masonry buildings indicate higher vulnerability when compared with brick masonry buildings. Note that due to the similar construction practices in Canada and in the United States, Hazus capacity curves and displacement based fragility functions were retained for vulnerability modeling of the brick masonry buildings. On the other hand, for stone masonry buildings, which are not explicitly considered in Hazus, the capacity curves and displacement based fragility functions were those generated by Abo-El-Ezz et al. (2013).

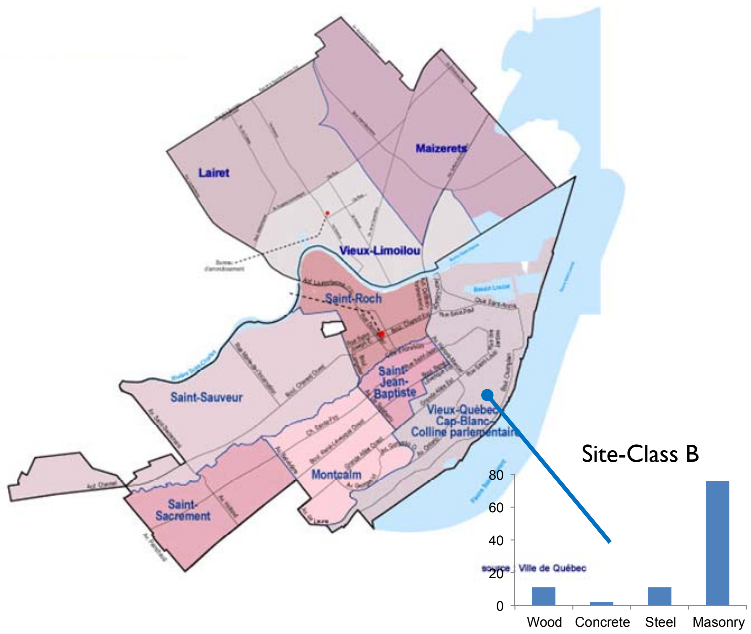

Figure 6. Map showing the Old Quebec City (Vieux-Quebec) and the results of the building inventory according to the structural material (Source: Ville de Quebec).

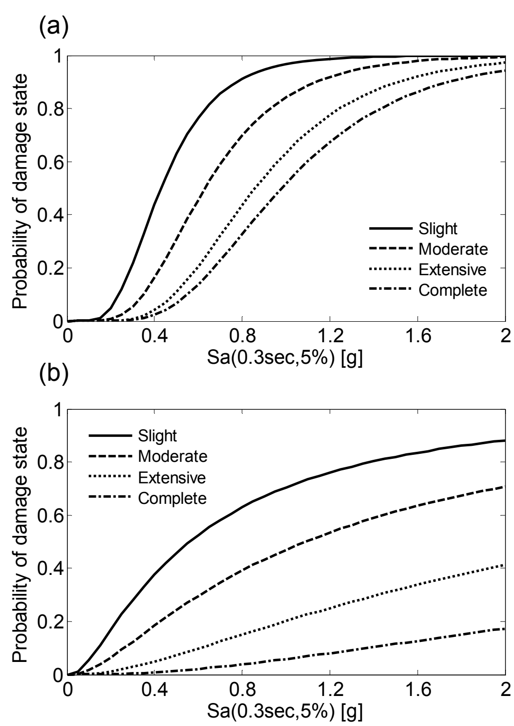

Figure 7. Applied fragility functions for (a) stone masonry buildings, and (b) brick masonry buildings.

The simulated damage to the 1220 buildings in the Old Quebec City for the considered M6.2R15 scenario is given in Figure 8. It can be expected that approximately 39% of the stone masonry buildings (65 buildings out of 168) and 33% of the brick masonry buildings (252 buildings out of 765) will be at least slightly damaged. These, relatively similar results, are a consequence of the similar threshold median capacities for slight and moderate damage states of the fragility functions for low IM (Sa0.3sec<0.4g) given in Figures 7. However, at higher IM, the difference increases (e.g. the stone masonry exhibits lower threshold capacities for extensive to complete damage).

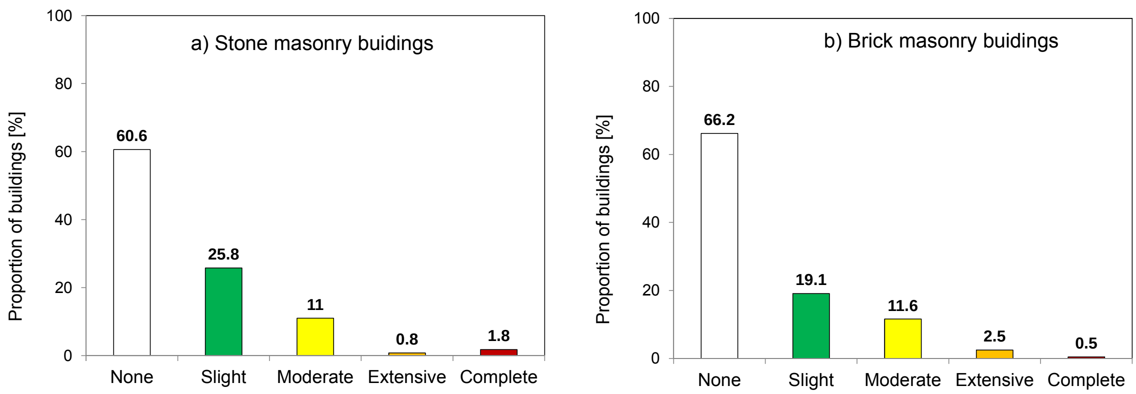

Figure 8. Proportion of buildings by construction material type in each damage state for a scenario event M6.2R15: (a) stone masonry buildings; (b) brick masonry buildings

CONCLUSIONS

A quantitative seismic risk assessment framework was validated for the existing unreinforced masonry buildings in the Old Quebec City. The framework includes: inventory of the structural characteristics of the existing buildings; seismic hazard definition using a ground motion prediction equation compatible to the seismo-tectonic settings and soil conditions in the study region to estimate the ground shaking intensity in terms of a structure-independent intensity measure IM (spectral acceleration at a particular period); and a vulnerability model represented as seismic hazard compatible fragility functions in terms of structure-independent IM. In this study Sa0.3s was selected as IM as it reflects the vibration periods of the masonry buildings, predominant in the considered study area. Capacity curves were applied to characterize the nonlinear structural behavior and the expected damage was assessed with displacement fragility functions. The developed analytical fragility functions were compared with reported empirical fragility functions for stone masonry buildings. The observed differences are due to different assumptions and tools used in the development process. These differences emphasize the complex interaction between the various input parameters and the need for development of fragility functions that reflect the site specific construction characteristics and seismotectonic settings. The collected inventory consisted of 1220 buildings. The damage assessment was performed for hypothetical scenario earthquake of magnitude 6.2 and hypocentral distance of 15km corresponding roughly to seismic event with probability of exceedence of 2% in 50 years. The simulated results indicate that approximately 39% of stone masonry and 33% of brick masonry buildings would suffer various levels of damage. Due to the relatively short computation time, the developed methodology is particularly useful for rapid regional-scale damage assessment studies.

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