VULNERABILITY ASSESSMENT OF EXISTING MASONRY BUILDINGS IN MODERATE SEISMICITY AREAS USING EXPERIMENTAL TECHNIQUES

C. MICHEL[1], E. LATTION, M. OROPEZA and P. LESTUZZI

ABSTRACT

Vulnerability assessment in moderate seismicity areas needs particularly accurate estimations of low damage grades, since they cause a great part of economic losses. For moderate level of shaking, the most important parameter is the fundamental resonance frequency as it varies with amplitude with consequences on seismic demand. In order to accurately but simply model the response of existing buildings to moderate earthquakes, in situ and laboratory tests together with a frequency drop approach were used. The dynamic behaviour of existing buildings, for which it is difficult to obtain plans and material physical properties, is first assessed by recording ambient vibrations simultaneously at several points. Resonance frequencies, damping ratios and modal shapes are derived using the Frequency Domain Decomposition method, a simple but efficient modal analysis method that allows decomposing modes, even close. These modal parameters, used in a linear multiple degree-of-freedom model, give the building response to weak ground motions. Additionally, an amplitude-frequency relationship has been derived from pseudo-dynamic tests of full-scale unreinforced masonry (URM) structures with reinforced concrete (RC) slabs made by ELSA laboratory in Ispra (Italy). This relationship is used in the MDOF model that can then be turned into a non-linear elastic model that should be able to well represent maximal response until moderate damage. Then, a 164 ground motions database selected in the European database is used to compute fragility curves, assuming inter-story (I-S) drift limits extracted from the literature and laboratory tests.

In this study, two URM buildings with RC slabs, typical of Swiss construction between 1940 and 1960 located in Visp (Valais) have been tested using full-scale ambient vibrations. A simple non-linear elastic model based on these tests and on laboratory pseudo-dynamic tests is built and used to compute fragility curves from slight to severe damage. The results are compared with displacement-based computations and numerical models.

[1] C. MICHEL, Applied Computing and Mechanics Laboratory, Ecole Polytechnique Fédérale de Lausanne, EPFL-ENAC-IIC-IMAC, Lausanne Switzerland
E. LATTION, Ecole Polytechnique Fédérale de Lausanne, EPFL-ENAC-SGC, Lausanne Switzerland
M. OROPEZA, Applied Computing and Mechanics Laboratory, Ecole Polytechnique Fédérale de Lausanne, EPFL-ENAC-IIC-IMAC, Lausanne Switzerland
P. LESTUZZI, Applied Computing and Mechanics Laboratory, Ecole Polytechnique Fédérale de Lausanne, EPFL-ENAC-IIC-IMAC, Lausanne Switzerland

INTRODUCTION

With a design acceleration ranging from 0.6 to 1.6 m/s^2, Switzerland is considered as a moderate seismicity area. However, the risk consciousness of the authorities is quite high since the Katanos (1999) and Katarisk (2003) reports that showed the earthquake risk in Switzerland was the greatest among all the disastrous events in terms of money. Moreover, Basel earthquake (1356) is the greatest historical event for North-Western Europe and new computations led to 6000 to 22000 fatalities in case of a similar event (Wyss et al., 2007). The highest hazard zone is located in Valais, where additional investigations led to a microzonation with large values up to 8 m/s^2 on the plateau of the design spectra in the Rhone valley (Crealp, 2005; Havenith et al., 2009). The COGEAR project (Fäh et al., 2008) is mostly dedicated to multi-risk hazard estimation in this area, and particularly around the city of Visp. However, there is lack of knowledge in the seismic vulnerability assessment of typical Swiss buildings. A quick survey showed that 2/3 of the structures in the city were unreinforced masonry (URM) structures (half from stone, half from brick masonry), 28% of RC shear wall buildings and 5% of wooden chalets. The use of empirical vulnerability functions (Risk-UE, 2003; Guéguen et al., 2007) could not reproduce the damage pattern of the 1855 I_{max}=VIII Visp earthquake (Fritsche et al., 2006) that strives for development of more accurate risk assessment methods towards more realistic fragility functions.

Modern displacement-based assessment methods offer the great opportunity to really take the physics of the problem into account and particularly modern estimation of the hazard (microzonation). For URM structures, Lang and Bachmann (2003) developed a simplified method that gives consistent results and that is used in the following. Numerical modelling of URM has been greatly improved by the development and implementation of macro-element models (Galasco et al., 2004), e.g. in the 3Muri software. The major issue of these methods is still the values of input parameters which estimation can be very tricky for existing buildings.

Starting from the observation that fundamental period was the key-parameter for low damage estimation, whereas these methods assumed a constant value, ambient vibration recordings were used to obtain the linear dynamic behaviour of existing buildings and extrapolate this behaviour to the non-linear domain by assuming a frequency amplitude relationship. This relationship has been obtained from laboratory pseudo-dynamic tests. The method is illustrated in this paper with 2 URM buildings located in Visp (Valais).

STUDY-BUILDINGS

In this paper, the case of two similar URM masonry buildings located in the city of Visp (Valais) is considered (Figure 1). These are 3-story detached houses with RC slabs referred in the following as A and B built in 1964 and 1959, respectively. The last story is a converted attic and the basement is half-buried. Their dimensions are approximately 22 by 9 meters with a 2.6 m story height. The façade and inner clay brick masonry walls are 15 cm thick. Building A has an additional 18 cm-thick transverse firewall but fewer walls in the longitudinal direction.

Displacement-based computations using Lang method and numerical modelling using 3Muri have been concentrated on building B. The main parameters used are the masonry characteristics and the reduction factor applied to the stiffness to account for cracking (0.5), the height of zero moment, taken as 1 story height. For Lang method, 7 and 9 walls have been considered in the longitudinal and the transverse directions, respectively. Period values are found significantly lower for Lang methods (Table 1), whereas a Rayleigh coefficient assuming a cantilever beam behaviour give the same results as 3Muri in that case.

Figure 1. Building A (left) and Building B (centre: picture; right: 3Muri model)

The buildings are located in the "Rhone Valley" area of the microzonation (Crealp, 2005) having a spectral acceleration of 6.3 m/s^2 on the plateau, where the elastic fundamental periods of the structures are. The displacement demand is then $S_d = \frac{6.3}{4\pi^2}T^2$ and depends much on the exact period value of the structures.

The displacement capacity obtained from the capacity curves obtained using both methods is displayed in Table 1. As the fundamental period in Lang method is shorter, the yield displacement d_y is much lower than 3Muri, especially in the longitudinal direction. The ultimate displacement capacity is coherent regarding the first wall that collapses (DG4) but the building collapse (DG5) occurs at quite different values.

Table 1. Computed periods and displacement capacity using Lang method and numerical modelling for building B

Direction	**Period (s)**		**Displacement (cm)**		
	Lang	**3Muri**		**Lang**	**3Muri**
Longitudinal	0.24	0.38	d_y^x	0.22	0.9
			d_u^x (DG4)	1.14	2.22
			d_u^x (DG5)	1.14	6.12
Transverse	0.21	0.29	d_y^y	0.23	0.5
			d_u^y (DG4)	1.15	1.30
			d_u^y (DG5)	1.15	2.69

EVOLUTION OF FUNDAMENTAL PERIOD

In moderate seismicity areas, there is a need not only for appropriate estimation of human losses that may be quite rare, but also for accurate estimation of low damage grades that may cover the greatest economic losses. The key-parameter governing low damage grades is the fundamental period of structures that controls the seismic demand. Non-linear static procedures like Lang method or 3Muri software consider a constant seismic demand related to the so-called "elastic period" and therefore do not always perform well for low damage grades, whereas their relevancy has been shown for severe damage to collapse.

The real linear fundamental frequency can easily be obtained using in situ ambient vibration records but cracks in masonry induce a decrease of this frequency when amplitude increases in both elastic and plastic domains (Dunand et al., 2006; Calvi et al., 2006). Thus, a quantification of this frequency decrease is necessary using laboratory experiments.

Therefore, laboratory pseudo-dynamic test made by ELSA laboratory in the Joint Research Centre of the European Commission (Italy) (ESECMaSE, 2008) were interpreted in terms of fundamental frequency drop (Michel et al., forthcoming). This type of experimental tests provides the opportunity to estimate the fundamental frequency at several time steps. The test unit is a 2-story clay brick masonry structure with RC slabs representative for traditional housing in North-Western Europe.

The ratio between the instantaneous frequency and the frequency at low amplitudes at the beginning of each test from 2% to 22%g is displayed on Figure 2 as a function of the inter-story (I-S) drift. The major result of this figure is that whatever the initial damage level, despite an important scatter, the shape of the frequency decrease remains approximately the same. This would indicate that low-level vibration frequency includes already the major part of the damage information. A tri-linear regression is proposed in Figure 2 as a frequency decrease model for this construction type.

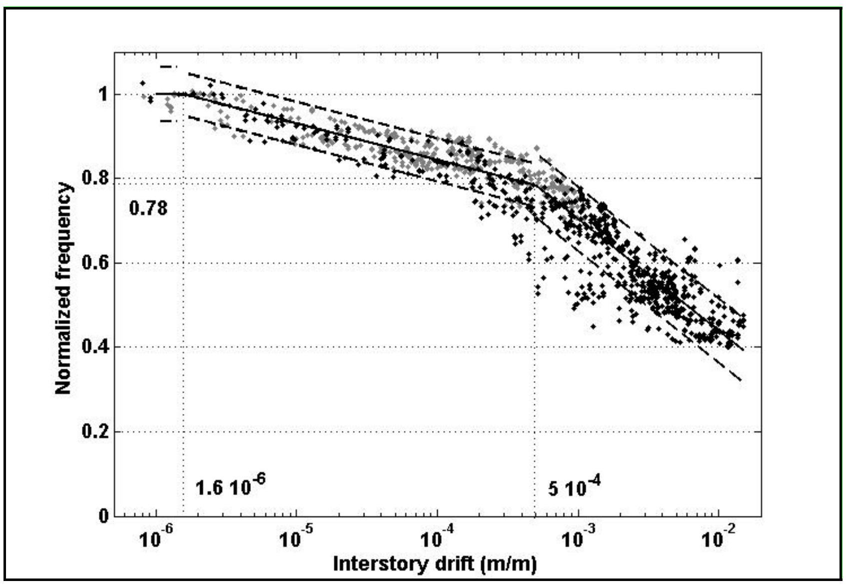

Figure 2. Frequency versus amplitude normalized by the initial frequency from ELSA tests on clay masonry structures. Grey dots: tests 2%g to 10%g, black dots: tests after 12%g. Trilinear fit of this dataset (solid line) with 80% confidence interval (dashed lines).

During the first part (I-S drift lower than D_0=1.6 10^{-6}), considered as the ambient vibration range, the frequency stays constant. After this point, the frequency starts to decrease slowly, without apparent damage. The maximum elastic frequency decrease is 22%. The major kink in the curve, at D_1=5 10^{-4} occurs before the apparition of the major crack in the building during the 12%g test. This first damage drift can be compared to the concrete tensile strength (10^{-4}). It could be related to EMS damage grade 1 (Grünthal et al., 1998), which is generally not considered because it does not imply retrofitting. During the 12%g test, the serviceability limit, i.e. damage grade 3 EMS, is reached. It occurs between 2 and 5 10^{-3} I-S drift, which is in accordance with the value 3 10^{-3} given by Calvi (1999). After this first damage state D_1, the frequency decreases more rapidly indicating a continuous development of cracking in masonry until a 50% relative drop.

MODEL BASED ON EXPERIMENTAL DATA

Since low-level vibrations are valuable information that can be extended to earthquake amplitudes following the proposed frequency drop model, ambient vibrations were recorded in the two study buildings with the Swiss Seismological Service. 9 Quanterra 26 bits Q330 with 9 Lennartz 3D 5s sensors synchronized by GPS were used to record simultaneously 8 points in each buildings and 1 point in free field. The modal analysis of the recordings was done using the Frequency Domain Decomposition (Brincker et al., 2001). However, the low signal-to-noise ratio imposed the computation of Transfer Functions between the free field recordings and the building to find the fundamental frequencies. For building A, the first longitudinal and transverse modes are both found at 4.6±0.4 Hz (0.22±0.02 s). The corresponding modal shapes have been extracted and show a slight coupling with torsion and an important relative displacement of the foundation (about 10% of the top displacement). This is not taken into account in the following. Building B gives fundamental frequencies at 4.4±0.1 Hz (0.227±0.006 s) and 5.1±0.1 Hz (0.196±0.004 s) in the transverse and longitudinal directions, respectively. Modal shapes show pure bending modes. As expected, these frequency values are greater than those found by numerical modelling since they do not represent the same quantity. For building B, the drop between the ambient vibration and the model frequencies reaches 22% and 48% in the transverse and longitudinal directions, respectively. Compared to the PsD tests, the longitudinal frequency may have been under-estimated by the numerical model. On the contrary, periods given by Lang method are comparable with the values found under ambient vibrations and therefore probably too short.

A simplified 1D lumped-mass model with 3 degrees of freedom (DOF) at each story (translation and rotation) is built. The response {U(t)} at each floor and for each DOF takes the form:

$$\{U(t)\} = [\Phi]\{h(t)\} + \{\Delta\}U_S(t) \tag{1}$$

where [Φ] are the modal shapes, Δ the direction vector, $U_s(t)$ the input motion and h(t) verifies:

$$\forall j \in [1,N] \quad h''_j(t) + 2\xi_j\omega_j(t)h'_j(t) + \omega_j^2(t)h_j(t) = -p_j U''_S(t) \tag{2}$$

with ω_j the resonance angular frequencies, ξ_j the damping ratios and p_j the participation factors. The modal shapes are deduced from the experimental modal analysis, the damping ratios are set to the conventional value of 5%. They are supposed constant until severe damage, which seems a good approximation for low damage but less for severe. It is assumed that the resonance frequency follows the relationship deduced from laboratory test with the initial frequency obtained under ambient vibrations.

The response in terms of I-S drift can therefore be calculated using this model. It should be emphasized that it is a dynamic non-linear elastic model so that in the plastic domain, the equal displacement rule has to be recall to ensure the computed displacement is equal to the inelastic displacement. For that reason and since the frequency decrease is only relevant for low damage grades, the model is used only until severe damage.

Damage grades (DG) according to EMS98 are set in terms of I-S drift following the results of the PsD test and Calvi (1999) paper at 5 10^{-4}, 10^{-3} and 3 10^{-3} for DG 1, 2 and 3. The uncertainty on these values is large. The study of the yield drift for 41 masonry walls of various materials tested in Italy, Germany and Switzerland leads to a lognormal distribution with a median drift 1.2 10^{-3} and a standard deviation of 0.44.

FRAGILITY CURVES

In order to compute fragility curves, 1000 cases using the proposed model have been computed. Each case requires a ground motion, selected among 164 recordings of the European Strong Motion Database (Ambraseys et al., 2002), a mass matrix, modal parameters (frequencies, damping ratios and modal shapes), a frequency drop relationship and an eccentricity to consider torsion. These values are randomly generated from the experimental values taking their uncertainty into account.

The results of the computations are sorted with respect to their spectral displacement. It should be emphasized that the spectral displacement S_d is computed at the structure frequency that varies with the amplitude. The proportion of event of each S_d class exceeding a given damage grade is one point of the corresponding fragility curve. A lognormal cumulative density function is then fitted on the obtained points for each damage grade. It is determined by a median value that is the most probable value, comparable to deterministic computations and a standard deviation including the variability of the process.

The uncertainty on the damage grade limit is assumed as independent from these computations and the variances are therefore added afterwards. A value of σ=0.44, as found in the study of masonry database (see previous §), is chosen for DG2 and DG3. The uncertainty on DG1 is included in the frequency drop relationship variability. More investigations should be made on these values.

The results for building B in the worst direction (i.e. transverse) are displayed in Table 2. They are quite similar for building A as the Figure 3, representing the corresponding fragility curves, shows. Although the period matches quite well, the median values are more optimistic in this study compared to numerical modelling. This period is indeed too short for the equal displacement rule to be valid so that the plastic displacement is underestimated leading to optimistic values for DG3. However, DG1 and DG2 seem appropriate since the yield displacement of the numerical modelling falls in between these two values.

Table 2. Parameters of the fragility curves computed for building B

Building B	**DG1**	**DG2**	**DG3**
Median (cm)	0.38	0.83	2.54
Standard deviation	0.38	0.51	0.49
Period (s)	0.28	0.32	0.38

In terms of risk, considering the microzonation for a 475-year return period, the building has a probability of 100, 90 and 50% to exceed respectively DG 1, 2 and 3.

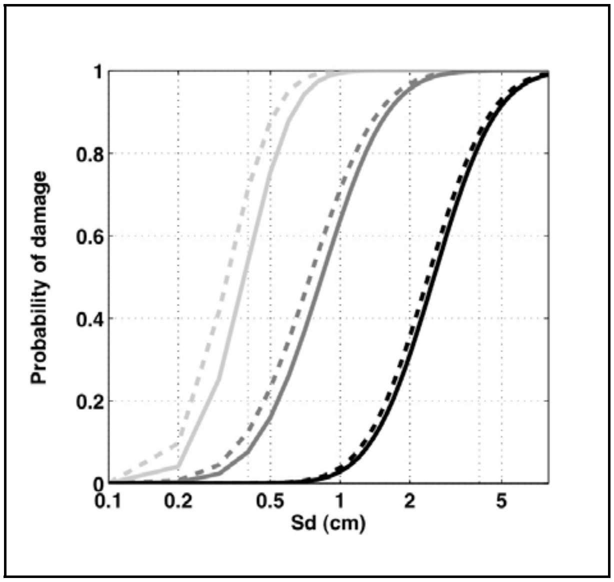

Figure 3. Fragility curves corresponding to damage 1 (light grey), 2 (grey) and 3 (black) of buildings A (dashed curves) and B (solid curves) computed according to the proposed method

CONCLUSION

An experimental frequency drop approach was followed to account for the fact that resonance frequency is the key-parameter that drives the response of a structure that is not too much damaged. The comparison with a simplified displacement-based approach and a numerical model showed important discrepancies: the first method underestimated the elastic periods leading to too small displacement capacity. The numerical model gives consistent period values but shows that the severe damage grade may be over-estimated in the proposed method. Therefore, a coupling of the proposed method and a mechanical method is necessary to obtain all the fragility curves. More care should be taken to the estimation of variabilities and uncertainties to estimate the standard deviation of the fragility curves. It will then be possible to draw realistic standard capacity curves for this building type. The same approach may also be used for RC structures using other frequency-amplitude relationships.

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