16th World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 4454
Registration Code: S-Y1463234074

PRELIMINARY ESTIMATION OF THE PROBABLE MAGNITUDE OF KOMÁROM 1763 EARTHQUAKE USING FRAGILITY FUNCTIONS

E. Morais[1], L. G. Vigh[2], J. Krähling[3]

[1] PhD Student, Budapest University of Technology and Economics, eduardo.charters@gmail.com
[2] Associate Professor, Budapest University of Technology and Economics, vigh.l.gergely@epito.bme.hu
[3] Associate Professor, Budapest University of Technology and Economics, krahling@eptort.bme.hu

Abstract

Pannonian Basin can be characterized as a moderate seismic region. One of the biggest known earthquakes in Hungary, which is located in the inner part of Pannonian Basin, occurred in Komárom in the 28th June of 1763. As result 91% buildings in Komárom were affected revealing damage states between light damage and total collapse. Surrounding cities were also affected as historical documental evidence supports. The earthquake was later characterized with uncertain magnitudes between 5.7 a 6.5, but the applied methods rely on the Modified Mercalli Intensity and then on empirical relationships to determine magnitude, completely disregarding the structural response analysis of real structures. Another estimation of the probable intensities of historical seismic events can be achieved if the behavior of real structures through dynamic structural analysis. The method exists and consists on modelling the structural damage due to the historical earthquake as a probabilistic event, where the relationship between the structural damage and ground motion intensity, magnitude and distance is represented by fragility functions. A version of this methodology is developed, validated and then applied for parametrical studies and preliminary estimations of the magnitude of the 1763 Komárom earthquake. To do so architectural archetypes are developed by combining structural seismic damage sources with historical surveying on historical buildings. A typical commoners' house structural archetype is generated and modelled as 2D portal frame macro-models using OpenSees code and calibrated to simulate the in-plane shear behavior of historical adobe masonry infill walls. Incremental dynamic analysis is performed together with 30 site-specific ground motion records in order to generate analytical fragility functions for reported damage states. Historical damage reports are combined with structure specific probability-of-damage estimates and the sensitivity of the method is studied with plausible intervals of the damage estimates and different attenuation relationships. The posterior distribution of magnitude given a damage event is estimated using Bayes' theorem. The formal framework for the use of prior distributions of distance and magnitude is presented for future developments. The framework of assumptions is tested in order to evaluate the relevance in reducing uncertainties as the calibration of the material and numerical models, material and geometric nonlinearities, the number and quality of structures falling into certain damage measures, and attenuation relations. A possible set of assumptions is interpreted from historical sources in order to assess the probable magnitude of the Komárom earthquake.

Keywords: Magnitude estimation; 1763 Komárom earthquake; Historical Structures; IDA; Fragility Functions;

1. Introduction

In regions with low or moderate seismicity, the amount of information characterizing the local seismicity is generally scarce, as well as the number of catalogued earthquakes. Therefore, historical earthquakes, occurred before the instrumental era in seismology, play an important role in determining the design ground motions and in seismic hazard analysis (SHA) [1]. The traditional methods for magnitude estimation rely on the document based historical descriptions of the damage caused by these seismic events to assign an intensity value using a scale such as the Modified Mercalli Intensity (MMI). Afterwards, they use empirical relationships either between the magnitude and epicentral intensity or between the magnitude and isoseismal area or radius, in order to calculate the magnitude. Such methods were used to estimate the intensity and then the magnitude of one of the most destructive seismic events in Hungarian history, with epicenter near the city of Komárom (northwest of Hungary). It occurred in the 28th June of 1763 and its intensity estimates vary from 8~9 [2], 9 [3] and 9.5 [4], the magnitude estimates from $M_w=5.70 \pm 0.34$ [5, 6, 7], 6.10 according to [3], 6.20 [8], 6.30 [9] and *6.50* [4] and distance to epicenter maybe between 10 and 12 km from the city of Komárom [1, 4]. These estimations show considerable dispersion if we consider that they are used in peak ground acceleration (PGA) based hazard map in national standards of the EC8 [1, 10].

A probabilistic method to estimate the magnitude of historical earthquakes using experimental and numerical studies of earthquake-damaged structures exists [11] and uses the background of Probabilistic Seismic Hazard Analysis (PSHA) [12], to generate fragility functions associated to the damage states and structures described in historical damage records. The fragility functions gives us the probability of a certain damage state or measure given an intensity measure (IM) [13]. Its parameters are usually estimated using either pushover nonlinear analysis or Dynamic Structural Analysis (DSA), either by Incremental Dynamic or Multiple Stripes Analysis (IDA or MSA, respectively) [13, 14, 15]. Afterwards, the structural fragility data is combined with the distribution of ground motion and prior distribution of distance using the total probability theorem in order to have the probability of the damage event given magnitude. Then, to calculate the posterior distribution of magnitude given the damage event the Bayes' theorem is used together with a prior distribution of magnitude (see Fig. 1).

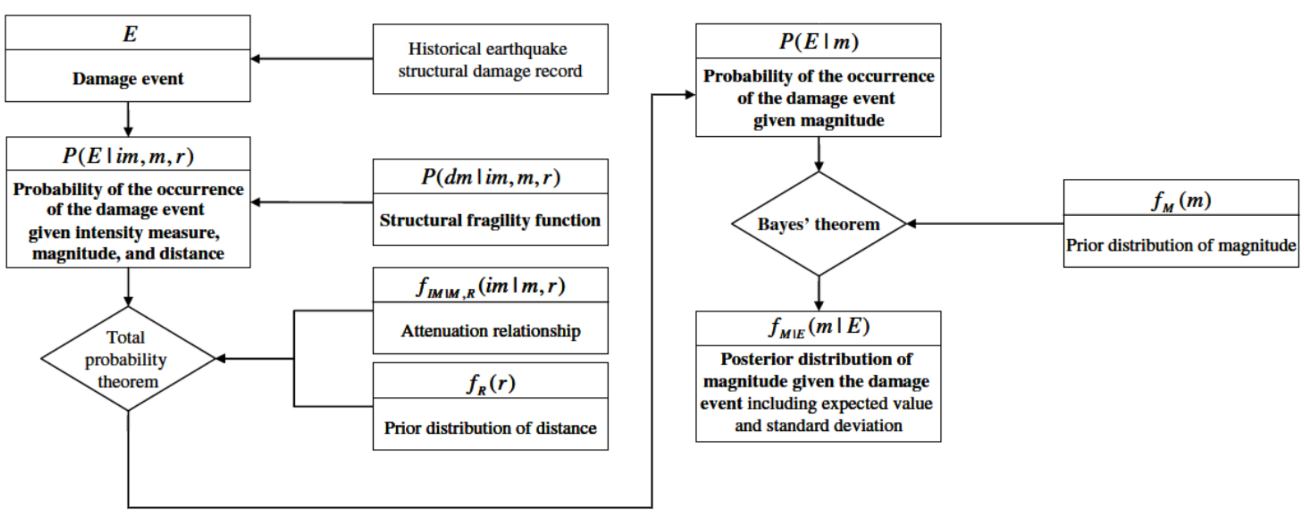

Fig. 1 – Flowchart of the probabilistic method for the magnitude estimation, from [11].

In the present paper preliminary magnitude estimations of the Komárom earthquake of 1763 are assessed. Firstly, in chapter 2, an outline of the formal framework of the method will be presented. Afterwards, in chapter 3, historical seismic damage records will be used together with historical architectural surveys to define historical relevant structural typologies for analytical fragility functions generation. The fragility outputs are also studied parametrically, considering damage states. In chapter 4 the formal framework of the method is implemented, validated and applied to the parametrical study of the historical archetype together with different attenuation relationships and prior distributions of distance and magnitude. Preliminary conclusions are to be drawn from the parametrical studies and later estimations.

2. Methodology framework

The earlier mentioned magnitude estimation methodology was developed and presented in [11] its outline can be followed in the flowchart in Fig. 1. The mathematical expressions relevant for present purposes are presented subsequently.

2.1 Structural fragility functions

Fragility functions give us the probability of a certain damage state *DS* given an intensity measure *IM*. They can be generated fitting the damage points determined using Dynamic Structural Analysis (DSA) (either IDA or MSA) or using Nonlinear Pushover Analysis, for example [16]. An IM can be a spectral acceleration (Sa) such as PGA, but also peak ground velocity, arias intensity, significant duration, among others. Assuming a lognormal distribution for fragility fitting, the general expression can be defined as a function of median $\mu_{IM,ds}$ and standard deviation $\beta_{IM,ds}$ in expression 1, where Φ is the normal CDF.

$$P\left(DS \geq ds \mid IM = im\right) = \Phi\left(\frac{\ln IM - \ln \mu_{IM,ds}}{\beta_{IM,ds}}\right) \tag{1}$$

Once the basic expressions are defined using expression (1), the probability of matching a damage state (DS) or damage measure (DM), defined by the set dm_k={dm_0 … dm_{nd}}, given an intensity measure can be calculated using probability expressed (2). Where dm_0 denotes no damage occurrence and $dm_{i>0}$ the occurrence of pre-defined structural damage measures, or states.

$$P\left(DM = dm_k \middle| \mathrm{im}\right) = \begin{cases} 1 - P\left(DM \geq dm_1 \mid im\right) & k = 1 \\ P\left(DM \geq dm_k \mid im\right) - P\left(DM \geq dm_{k+1} \mid im\right) & 1 \leq k \leq n_d - 1 \\ P\left(DM \geq dm_{n_d} \mid im\right) & k = n_d \end{cases} \tag{2}$$

Afterwards, the probability of occurrence of the event given intensity measure *P(E|IM,M,R)* (and also magnitude and distance) can be calculated combining the damage information prevenient from seismic damage historical records associated to the damage states using the general expression (3).

$$P\left(E \middle| \mathrm{IM{=}im}, M = m,\ R = r\right) = \prod_{i=1}^{n_s}\left[\sum_{j=1}^{n_e^i} \frac{n_t^i\,!}{\prod_{k=0}^{n_d^i} {}^j n_k^i\,!} \times \prod_{k=0}^{n_d^i} P\left(\mathrm{DM{=}}dm_k^i \middle| \mathrm{im}, m, r\right)^{{}^j n_k^i}\right] \tag{3}$$

Besides the number of structures (n_s) and related damage measures (dm_k) that can be derived from historical data, it is important to define the number of structures falling into each of the damage measures, denoted by the array n_k=[n_0 … n_k], and the total number of structures $n_t = \Sigma n_k$.

2.2 Total probability theorem

The probability of the damaging event given magnitude uses the total probability theorem to combine the uncertainties relating structure/damage and intensity measure, and in distribution of distance.

$$P\left(E \mid m\right) = \iint P\left(E \mid \mathrm{im}, \mathrm{m}, \mathrm{r}\right) \times f_{IM|M,R}\left(im \mid m, r\right) \times f_R\left(r\right)\, \mathrm{dim}\, \mathrm{d}r \tag{4}$$

Here, the attenuation relationship in expression (5) is used to express the intensity measures presented both in expression (1) and (4). In the former it helps to build the fragility functions, in the later it simplifies the distribution of intensity measure, which can be simply be described as the normal distribution of ε.

$$\ln\left(IM\left(M, R, \Theta\right)\right) = \ln\left(\overline{IM}\left(M, R, \Theta\right)\right) + \sigma_{\ln IM}\left(M, R, \Theta\right) \cdot \varepsilon \tag{5}$$

The methodology [11] provides several options to compute the prior distributions of distance as can be seen in Table 1. The options vary from general (column 1) to more complex and piecewise defined (columns 2 and 3). The sensitivity of the method to different prior distribution types will be tested in subsection…

Table 1 – Prior distributions of distance f_R (row 1) and magnitude f_M (row 2).

Distrib. Functions	**1**	**2**	**3**
$f_R(r)=$	$\frac{2}{r_u^2}r$	$\frac{8}{\pi r_u}\left(\theta\sin^2\theta\cos\theta-\sin\theta\cos^2\theta+\theta\cos^3\theta\right)$	$2r/r_b^2$, $\frac{r}{\pi}\left(\frac{2\alpha-\sin 2\alpha}{r_b^2}+\frac{2\beta-\sin 2\beta}{r_a^2}\right)$
$f_M(m)=$	$\frac{1}{m_u-m_l}$	$\frac{c}{m\varsigma\sqrt{2\pi}}\exp\left(\frac{-\left(\ln m-\lambda^2\right)}{2\varsigma^2}\right)$	$\frac{2.303\times b\times 10^{-b(m-m_l)}}{1-10^{-b(m_u-m_l)}}$

2.3 Bayes' theorem

The result of the disaggregation with the Bayes' theorem is the posterior distribution of magnitude given the damage event, with expected value of magnitude and standard deviation, in expression (6). This expression uses a prior distribution of magnitude (Table 1) that can vary from a uniform type (column 1) to more complex, as the truncated Gutenberg-Richter (column 3). The method sensitivity to these distribution types will also be evaluated.

$$f_{M|E}\left(m|E\right)=\frac{P\left(E|m\right)\times f_M\left(m\right)}{P\left(E\right)}=\frac{P\left(E|m\right)\times f_M\left(m\right)}{\int P\left(E|m\right)\times f_M\left(m\right)\,dm} \tag{6}$$

3. Fragility functions for historical typologies

The Komárom earthquake of 1763 is relatively well documented by contemporary sources and most of these historical documents can be traced back by later sources such as [2, 4, 17, 18, 19]. This historical seismic event is taken in the present paper to exemplify the study of the historical macro-seismic effects, endorse structural typologies, modeling strategy and DSA, in order to generate historically based structural fragility functions.

3.1 Historical survey

Contemporary sources of the 1763 Komárom earthquake are composed by a letter to the Empress Maria Thereza, four depictions of the city of Komárom (two of them in Fig. 2), official reports of the damage and on the costs for tax-payers, and references in Johann Grossinger's dissertation [20]. Despite some remarks on the reliability of the depictions [19] later sources objectively state that out of 1169 houses (91% of the total) in total, 279 ended up completely destroyed, 353 partially collapsed, 213 needed expensive repair and 219 cheap repair [3, 4, 17]. The great majority of the buildings (perhaps all the city houses) were typically made of masonry walls (stone, clay or adobe bricks with mortar infills) arches, vaults and/or wooden roofs. The majority of the houses of the town belonged to local peasants and were built of adobe or mud wall [21] possibly built with flexible willow-twigs.

Fig. 2 – Depictions of the damage in Komárom after the 1763 earthquake: a) Karl Friedl’s depiction and b) unknown author’s depiction [18, 19].

The foundation structure on which the walls were built was just the well-rammed level of soil. The typical commoners’ houses (Fig. 3b) were built with open chimney covering the kitchen in the middle therefore there were smoke-free living rooms as their stoves were heated from the kitchen. Their typical layout as seen from the street consists of room, entrance hall with doors to the rooms and connected to the kitchen, kitchen, room and pantry. The houses had beam ceiling, a gable roof or a hip roof, with thatch-roofing or covered with straw. Parallels to the rural houses can be seen at model plans of the region [22]. According to the records of the earthquake made by the eye-witness Johann Grossinger, the commoners’ houses with their simple structures and one-story showed more resistance against the impacts of the earthquake then the brick-built public, ecclesiastical or civil houses of the town with possibly multiple stories, more embellishment and vaulted constructions [20].

Once the contemporary sources describing the damage caused by the seismic event are gathered, there are two ways of dealing with the problem of relating the damage to the damaged structures. One way is to identify directly in the sources specific structures that are still standing or for which plans still exist. This is usually the case of monumental buildings, such as churches, castles, palaces or public buildings, which in most cases underwent major interventions or still exist as ruins. This would be the case of the Zsámbék church [23, 24]. A second option, adopted here, is to establish structural archetypes from common architectural forms of the period using historical descriptions and surveys, as well as expert opinions. In this way, 1 and 2 stories archetypes were composed among city and landscape houses (Fig. 3a). The most common dimension patterns and structural related features were also extrapolated from the investigation of the historical survey. It was observed that for most of the city buildings of the 18th c. (excluding some monumental buildings) span lengths varied between 3 to 2,50 fathom (1 fathom = 1,896m & 1/6 fathom = 1 foot = 0,316m). The heights varied between 2 and 2,50 fathom and the walls width between 2 and 2,5 feet. As to the overall structural typology, walls alone or together with vaults, gable roof with a typical baroque timber structure.

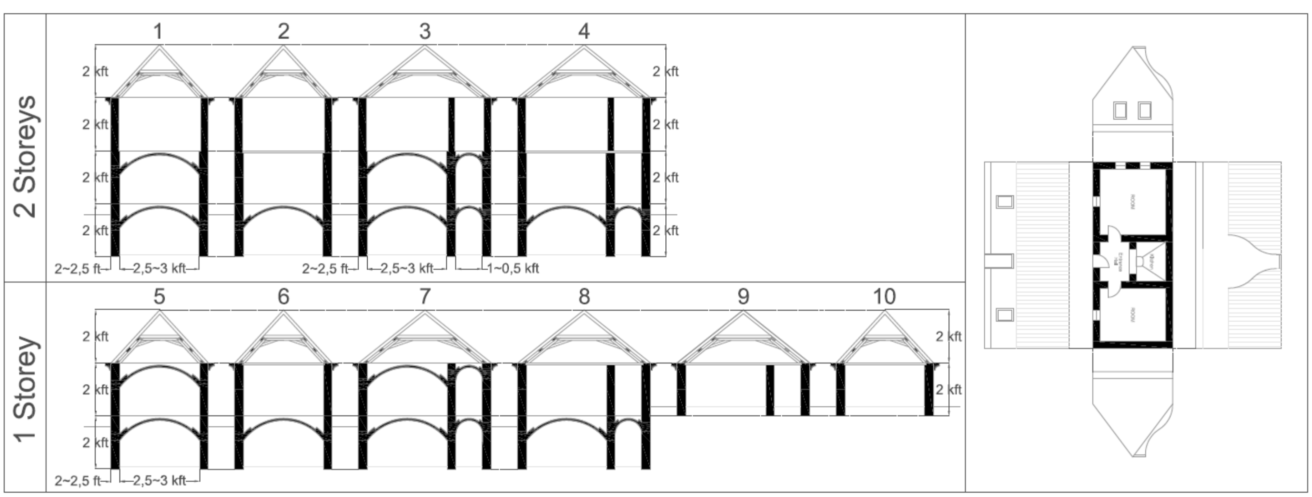

Fig. 3 – Architectural typologies: a) typical cuts & b) commoners' house archetype.

3.2 Dynamic structural analysis

In a previous study [25], IDA was implemented in *OpenSees* [26] interface, for the commoners' house archetype (Fig. 3b) adopting sensible values for adobe masonry walls' shear modulus $0{,}10{\cdot}E_w{=}G{=}39{,}80\ MPa$ and shear strength $f_v{=}0{,}026\ MPa$ [28, 29]. To model the material in-plane shear behavior it was applied a pinching4 material model which consists in a multi-linear spring (with a force-displacement based input – Fig. 4a), available in OpenSees [26]. The process was validated and then these values were generalized in order to generate macro-models (Fig. 4b) matching structural archetype specific characteristics and material damage measures (Table 2). To perform IDA 30 site specific ground motions (for the city of Komárom) were applied [27]. One of the conclusions of this work was that it was not likely that the masonry walls in commoners' houses were made of clay bricks, but rather of adobe, or perhaps a weaker material such as mud bricks.

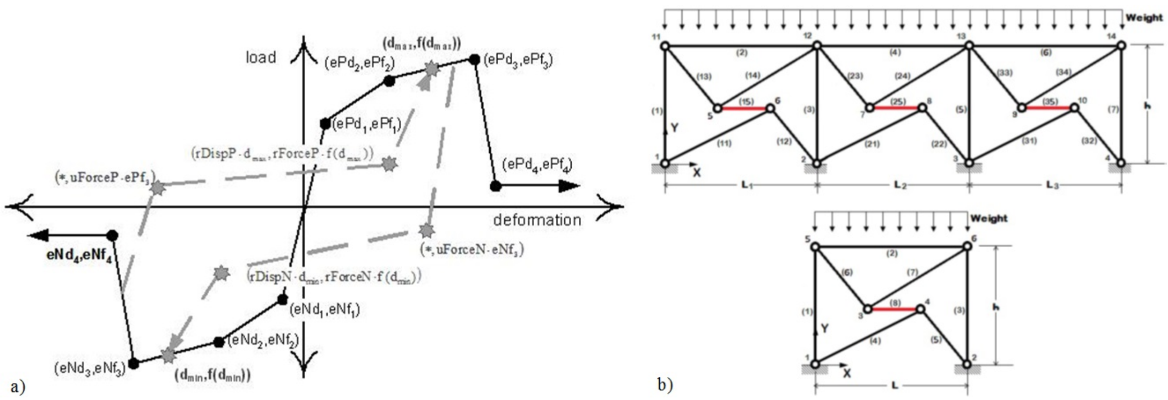

Fig. 4 – a) Pinching4 material model and b) basic structural macro-models.

Table 2 – IDA analysis: a) input parameters for pinching4 model (in fig. 4a), and b) fragility parameters for adobe walls.

		ePfi/eNfi[kN]				ePdi/eNdi[mm]			
	i=	1	2	3	4	1	2	3	4
Panels	1	42,9	53,63	64,35	5,36	1,5	3,01	10,91	16,81
	2	55,9	69,88	83,85	6,99	1,5	3,01	10,91	16,81
	3	68,9	86,13	103,35	8,61	1,5	3,01	10,91	16,81
	4	24,7	30,86	37,05	3,09	1,5	3,01	10,91	16,81
	5	37,7	47,13	56,55	4,71	1,5	3,01	10,91	16,81
	6	50,7	63,38	76,05	6,34	1,5	3,01	10,91	16,81

	front wall		back wall		lateral wall 1		lateral wall 2	
Panels	4 + 5 + 6		1 + 2 + 3		1		4	
DM=	**ln(μ)**	**σ**	**ln(μ)**	**σ**	**ln(μ)**	**σ**	**ln(μ)**	**σ**
dm1	0,119	0,237	0,248	0,214	-0,558	0,228	-0,910	0,230
dm2	0,380	0,179	0,052	0,168	-0,294	0,184	-0,598	0,212
dm3	0,560	0,130	0,246	0,130	-0,079	0,235	-0,287	0,215
dm4	1,950	0,134	0,391	0,176	0,086	0,188	-0,119	0,247

Despite results, the strength parameters show great variability [30] raising considerable uncertainty on the median $\mu_{PGA,ds}$ and standard deviation $\beta_{PGA,ds}$. Therefore, the issues of material modeling were revisited. The shear stress corresponding to damage measure 1, $f_{DM=dm1}$ was changed to one third of the ultimate shear stress f_u affecting both ePf_1 and eNf_1 values (in Table 2). Parametrical studies were carried out using IDA, with increments $0{,}01g$, to evaluate the sensitivity of the fragility parameters. This set of results followed experiments on traditional adobe walls [28] and varying shear strength from $f_v{=}0{,}020$ to $0{,}032\ MPa$ and shear modulus $G{=}15{,}30$ to $62{,}34\ MPa$.

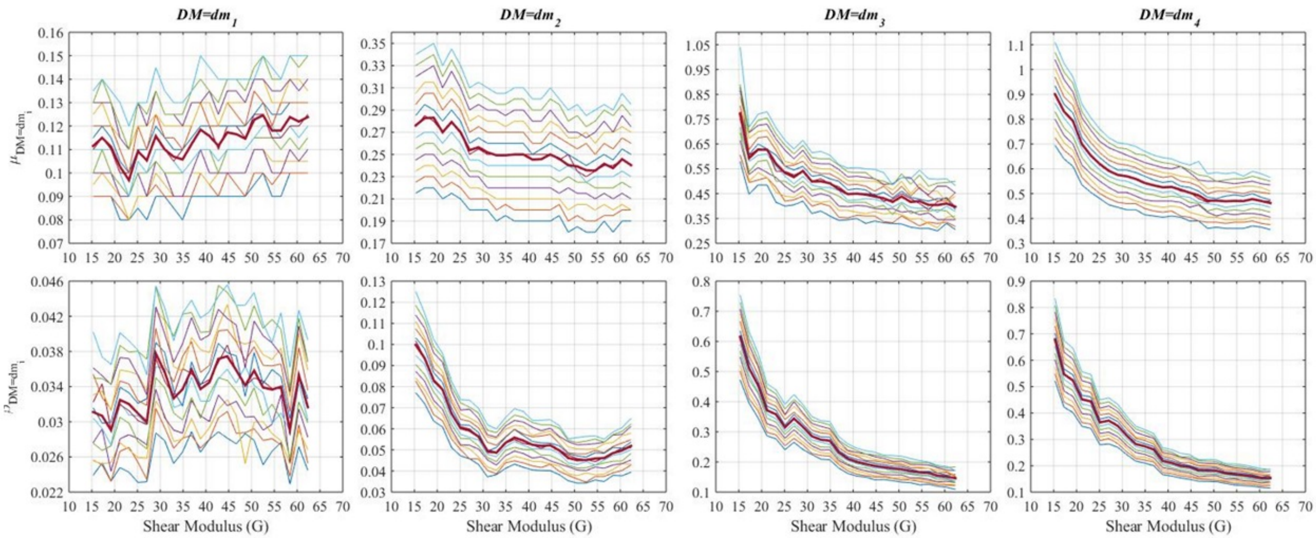

Fig. 5 – Parametrical study for $DM=dm_i$: variation of $\mu_{PGA,ds}$ (1st row) and $\beta_{PGA,ds}$ (2nd row) with shear modulus, for different shear strength levels from 0,020 to 0,032 MPa.

The results, for the all damage measures $DM=dm_i$ (in Fig. 4), show that both $\mu_{PGA,dm1}$ and $\beta_{PGA,dm1}$ values are direct and linearly proportional to shear strength. These values present some variation with the shear modulus, which increases with the damage measure (Fig. 4, from left to right). The averaged median and standard deviation for the damage measures 1 to 4 are $PGA_{dm}=[0,114\pm0,034 \quad 0,253\pm0,058 \quad 0.491\pm0,265 \quad 0,568\pm0,288]$. Globally, it can be said that these values show smaller variation for the first two damage measures, than for the latter two. This fact may be explained by the fixed increment in IDA, for it is advisable, for future estimations and in order to achieve a better description, to use increment values varying with the shear strength, for which the model showed higher sensitivity.

4. Preliminary studies on magnitude estimation

The present study is circumscribed by previous results on the magnitude estimation, as well as by other studies regarding the Komárom 1763 earthquake. Furthermore, previous estimations of the probable source type and location are also addressed, for distance between the city of Komárom and the epicenter is between 10 and 12 km [1, 4]. Such results are assumed here. Therefore, the expression (3) is simplified:

$$P\left(E\,\middle|\,\mathrm{IM{=}im}, M=m,\ R=r\right)=\frac{n_t\,!}{\prod_{k=0}^{n_d} n_k\,!}\times\prod_{k=0}^{n_d} P\left(\mathrm{DM{=}}dm_k\,\middle|\,\mathrm{im}, m, r\right)^{n_k} \tag{7}$$

Stating the distance with discrete values, eliminates the account for distance uncertainty, affecting also expression (4), leaving the estimation of $P_{E|pga,m}$ as a function of the PGA and magnitude. In this case the distribution of PGA is also described by expression (5) together with a given attenuation relationship.

$$P\left(E\,|\,m\right)=\int_{PGA} P\left(E\,|\,\mathrm{pga,m,r}\right)\times f_{\mathrm{PGA}|M,R}\left(\mathrm{pga}\,|\,m,r\right)\,\mathrm{dpga} \tag{8}$$

The method has been validated for the 1994 Northridge earthquake, as in [8]. The HAZUS technical manual [16] was applied for the determination of the fragility parameters. The Abrahamson & Silva attenuation relationship [31] was used to create the attenuation parameters and the distribution of PGA for the western United States. General prior distribution of distance and uniform prior distribution of magnitude were applied (Table 1). The most probable magnitude was confirmed with this algorithm as 6.8, matching the validation example in [11].

4.1 Uncertainty and sensitivity studies

The parametrical study conducted in 3.2 to evaluate the uncertainty in the fragility function parameters shows great variability in the PGA median estimates for damage states $DM=dm_i$, due to the variation in the strength and stiffness parameters implemented in the material model. Furthermore, the intensity estimates for the 1763 earthquake vary from VIII~IX to IX, the magnitude estimates from $M_w=5.70 \pm 0.34$ [6] to $M_w=6.50$ [3] and distance to epicenter between 10 and 12 km from the city of Komárom [1, 4].

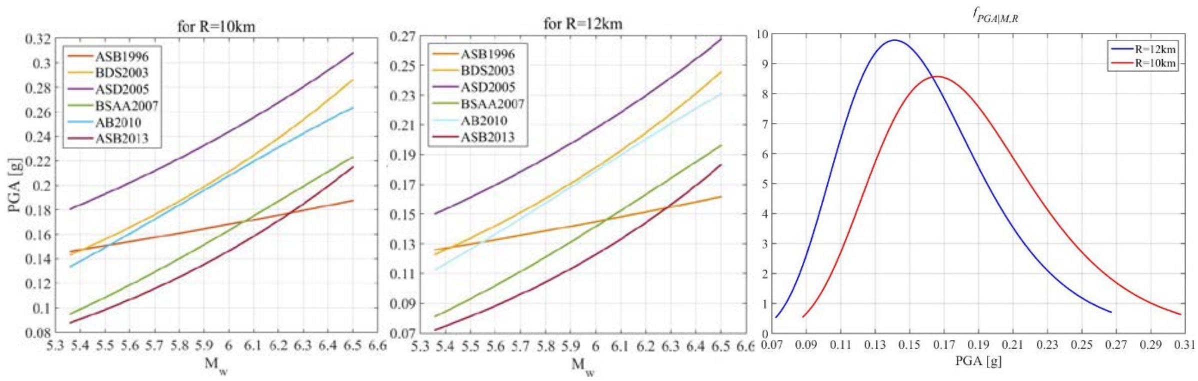

Fig. 6 – PGA plots for each of the attenuation relationships a) for R=10km, b) for R=12km, with 5.36≤Mw≤6.50, under soft soil conditions, & c) respective distribution functions.

Based on these values, some attenuation relations for Central Europe [32-37] (a relation is presented in Table 3), after being implemented and validated, they output a wide range of PGA values, from a minimum of 0,07g (Fig. 1b) to a maximum of 0,31g (Fig. 1a). The PGA outputs can be fitted assuming a lognormal distribution (Fig. 1c), with respective moments $\mu_{PGA|R=10km}=0{,}178 \pm 0{,}271g$ and $\mu_{PGA|R=12km}=0{,}153 \pm 0{,}278g$ (Fig. 1c). While the national annex for EC-8 indicates $a_{gR}=0{,}15g$ (zone 5) [10] which does not fall much out of the median values show great dispersion. The uncertainty related to the choice of one attenuation relation not accounted for in the methodology in [11], although the effect of different choices can be studied through the results.

Table 3 – a) Limits of applicability & b) relation of names for the attenuation relationships, from [37].

	ASB1996	BDS2003	ASD2005	BSAA2007	AB2010	ASB2013
Number of earthquakes	157	157	135	289	131	221
Number of records	422	422	595	997	532	1041
Horizontal component	Larger	Larger	Larger	GM	GM	GM
Minimum response period (s)	0,1	0,1	0,05	0,05	0,05	0,01
Máximum response period (s)	2	2	2,5	0,5	3	4
Magnitude scale	Ms	Ms	Mw	Mw	Mw	Mw
Mínimum magnitude	4	4	5	3	5	4
Máximum magnitude	7,9	7,9	7,6	7,6	7,6	7,6
Distance type	Repi	Repi	Rjb	Rjb	Rjb	Rjb, hyp, rup
Máximum distance (km)	260	260	99	100	99	200
Number of free coefficients	6	8	10	10	10	11

Source article	Ref.
Ambraseys et al (1996)	ASB1996
Bommer et al (2003)	BDS2003
Ambraseys et al (2005)	ASD2005
Bommer et al (2007)	BSAA2007
Akkar & Bommer (2010)	AB2010
Akkar et al (2013)	ASB2013

One great difficulty with a broad parametrical study of the method in [11] consists on the number of input parameters and their uncertainty. In the present paper just one type of structure is analyzed, although, its strength and lateral stiffness show considerable variability (subchapter 3.2). More, the numbers and measure of damages due to the seismic event were estimated, in the city of Komárom, but it is not certain how many of these structures were subjected to shear or shear-compression phenomena, and in which degree. To study these two major uncertainties would demand considerable computation time. Therefore, the studies are conducted separately: to study the uncertainty related to the historical seismic records average structural parameters are assumed, and to study the uncertainty related to the structural parameters, as well as the possibilities of different prior distributions of distance and magnitude (Table 1), a certain damage record is assumed.

4.2 The 1763 Komárom earthquake

The attenuation relationship AB2010 [36] has previously been used to estimate the distribution of accelerations in the Pannonian basin [24]. This relation, expression (5), was inverted $PGA(M,R,\theta) \rightarrow M(\mu_{dmi},R,\ \theta)$ in order to attain a very rough approximation of the magnitude of the 1763 earthquake using the median values calculated in 3.2. These estimations, for each damage state were averaged, assuming that each damage measure equally contributes to the magnitude estimates. The results ranged between 6.20 to 6.97, and 6.44 to 7.08, for distances of 10 and 12 km, respectively.

As mentioned before, historical records have references to the most and less affected typologies. Although, the nature of the remarks are vague for present state of the art, and therefore, a problem of how to transform qualitative to quantitative data characterized by an array of the type n_k=[ndm_0 ndm_1 ndm_2 ndm_3 ndm_4] arises. This is where the reliability and totality of historical descriptions steps into the methodology. Most of the damage can reliably be associated with shear failure modes, as can be seen in figure 2, but the number of structures falling into this category is still a question to be answered. A parametrical study was carried out to evaluate the impact of the different parameters in the n_k array. Following the methodology ndm_0 was fixed at 105, ndm_1 between 0 and 279, ndm_2 between 0 and 353, ndm_3 between 0 and 213 and ndm_4 between 0 and 219, with increments of 10. Despite some numerical issue, mainly because of the low values arising with powers of hundreds, results showed great dispersion, ranging between 5.3 and 7.5 of expected magnitude values. The sensitivity of the method to the parameters of the n_k array was confirmed.

In order to carry out the magnitude estimation, and despite equation (8), the distance uncertainty was fully incorporated using expression (4). Another integration step was added to the command lines. Furthermore, the previously mentioned attenuation relationships and presented in table 3 were applied in order to evaluate the variability of the results. A general distribution of distance (between 0 and 20 km) and a uniform distribution of magnitudes (between 5 and 8) were applied. As to the n_k array for the number of damages under each damage measure, the values were chosen arguably assuming the descriptions of Johann Grossinger [20], which state that less peasant houses revealed greater damage measures, and a percentage of approximately 20% of houses over the full building damage observed in the city of Komárom. The array was then inputted with 105 none collpases, 44 light damages, 42 medium damages, 35 severe damages and 27 total collapses, and therefore n_k=[*105 44 42 35 27*]. The results (fig. 7) show considerable variability. After probability and distribution calculations, the expected values of magnitude and standard deviation were calculated using the distribution functions (fig. 7b). The results, $M_{exp.}$=[6.68 ± 0.83 6.16 ± 0.57 6.03 ± 0.55 6.63 ± 0.57 6.53 ± 0.76 6.38 ± 0.80].

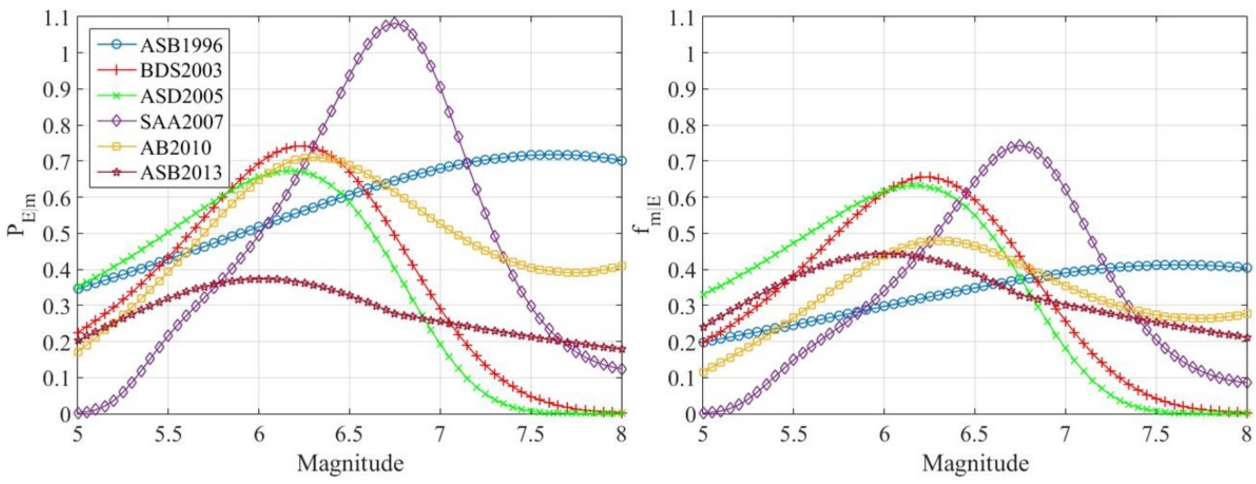

Fig. 7 – Estimates for a) $P_{E|m}$ and b) $f_{m|E}$ for different attenuation relations.

5. Conclusion

The estimation of the probable magnitude of historical earthquakes is a process dealing with uncertainties of different sources and nature. To improve the quality of the estimates using information on the historical damage in structures, not only the contribution of geophysicists and historians, but also of structural/earthquake engineers and architecture historians is important. Expertise in each field should lead to the characterization of regional structural, historical and seismic features. In the present paper, a preliminary study on the estimation of the magnitude of the 1763 Komárom earthquake was addressed. Differently from the first application of the studied methodology [11], the present study focused on a relatively more complex type of structural assembly, [adobe] masonry walls. Besides its regional complexities, due to material source and construction techniques, the failure modes are of difficult incorporation in a single simplified structural macro-model. However, shear-compression failure can be assumed as typical behavior mode of Hungarian commoners' house archetype. It is still a question if this archetype is a good candidate to represent the structures damaged by the 1763 earthquake, and if it is in which extent. Material related uncertainties were proven relevant for performance assessment, providing variability in IDA results.

There is a considerable discrepancy in previous magnitude estimations of the 1763 event, as it was seen in 4.1. This fact can be misleading in the context of PSHA [1] and it is yet to be evaluated. Two estimations are considerably incongruent: while the first, with M=6.20 [7] has [2] as source, the second, with M=5.8 [8] has [6] as source. This issue shall be addressed in future studies with a revision and comparison of the source methodologies. A common framework may be needed for that purpose.

Despite the consistent application of the validation example, from [11], some numerical inconsistencies appeared in the course of the algorithm application. Most of these are probably related to numerical limitations. Most of the estimations approach either very low or very high values, sometimes requiring symbolic treatment of variables. This fact did not affect the estimations of the expected values of magnitude between 6.0 and 6.7, which were the main objective of the present stage of the studies. One advantage of the method [11] is that the framework of assumptions can be tested and enhanced based on new improved (by verification) information. If the depicted sources (Fig. 2) pose some limitations on the reliability of the final assessment, it is important to survey on historical demography or income reports of the time. The development and deeper study of other archetypes and real structures (as churches, castles and palaces) that are still standing, or exist as ruins, and in other cities affected by the earthquake, may stress, outwit or complement some previous estimations. Seismic record selection was not addressed here, and it will be relevant to rethink how selection conditions may influence results. Also, the impact of different prior distributions of distance and magnitude was not stressed, which will be addressed in future studies. Nonetheless, a possible set of conditions was assessed. In order to strengthen the statement of probabilistic assessments of the magnitude of this earthquake, all major uncertainties sources shall be reviewed and the study extended to other structures.

6. Acknowledgements

Proper acknowledgements to CAPES for allowing the present research (process n^{o}. 9178-13-9). To Ádám Zsárnoczay, József Simon and Tamás Bálogh profound gratitude for making the research work plural and pleasant. We are grateful to Dr. Hyeuk Ryu for providing his article and make himself available to discuss the algorithm, Dr. Péter Varga for the seismic damage data and Mrs. Vida Mária for her assistance in the historical research. This paper was also supported by the János Bolyai Research Scholarship of the Hungarian Academy of Sciences.

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