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EXPECTED ECONOMIC LOSSES DUE TO EARTHQUAKES IN THE CASE OF TRADITIONAL AND MODERN MASONRY BUILDINGS

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Abstract

A seismic loss estimation methodology for masonry buildings is briefly presented and its use is demonstrated by estimating seismic losses for a traditional and modern masonry building. The seismic loss methodology is based on PEER probabilistic approach, where the problem is first decomposed in seismic hazard analysis, structural analysis, damage analysis and loss analysis. The results of the analyses are then convolved in a probabilistic manner, using the total probability theorem. The methodology makes it possible to communicate information about seismic risk by various performance measures such as the probability of exceeding a certain damage state, the probability of exceeding a certain economic loss, the expected annual loss, and, for example, the expected loss given seismic intensity. An emphasis is given on the structural and damage analysis which means that the economic loss is simulated directly from the results of structural analyses. Since such an approach can become computationally quite demanding, the pushover-based method was used for the estimation of engineering demand parameters. The proposed methodology is capable to take into account the effects of ground-motion randomness and the epistemic uncertainty. However, for simplicity reasons the use of the methodology is demonstrated only by consideration of the ground-motion randomness which is approximately accounted for by the incremental dynamic analysis of an equivalent SDOF model.

Seismic risk assessment was performed for two three-storey masonry buildings which have the same geometry but different quality of masonry with the aim to present the seismic risk of buildings built in various time periods. Results of the study indicated, that the median capacity of the modern building when expressed in terms of peak ground acceleration was almost 200 % higher than that of the traditional building. The probability of collapse in 50 years and the expected annual loss for the traditional masonry building were observed, respectively, 4.5 and 2.5 times higher than the results corresponding to the modern variant of the investigated building. The expected annual loss per 100 m^2 of gross floor area was estimated to 50 and 123 € for modern and traditional variant of the building. It was also found that non-structural elements are the key components in the loss assessment model since they contribute more than 50 % of the total loss.

Keywords: traditional and modern masonry; seismic risk assessment, loss estimation; probability of collapse; expected annual loss

1. Introduction

Several approaches for loss estimation were developed in the last decade [1-4]. Among others, Pacific Earthquake Engineering Research Center (PEER) developed a probabilistic framework [5], which enables the loss estimation and at the same time propagation of uncertainties through four independent parts of the methodology: hazard analysis, structural analysis, damage analysis and loss analysis. Monte Carlo simulations are used to compute losses given the statistical model of seismic response of a structure.

The motivation of this study was to simulate losses directly for each seismic response analysis and to investigate the impact of material quality on the loss estimation in the case of masonry buildings. The PEER methodology was used as the basis for this study, but it was applied in a different manner than suggested in FEMA P-58 [5]. The loss estimation methodology, as presented in this paper, enables explicit consideration of ground-motion randomness and the modelling uncertainty and the estimation of losses based on actual demand and damage obtained from structural analysis. For simplicity, only the effect of ground-motion randomness is considered in examples.

The complexity of the structural analysis is, according to the opinion of the authors, one of the main reasons that the loss estimation methodology is rarely applied to masonry buildings [6, 7], which represent majority of building stock in Europe. However, it was shown elsewhere [8, 9], that the pushover-based methods can provide sufficiently accurate results in the case of masonry buildings.

In this paper, firstly the methodology for loss estimation is briefly presented and its application is demonstrated by means of two examples of a three-storey masonry building made from a traditional masonry of lower quality and modern masonry.

2. Loss estimation methodology

In this study the PEER methodology for loss estimation was used in a different manner than it was suggested by others [1-5]. In the modified methodology [10], the damage and losses are assessed for each simulation from the results of the seismic response analysis. Such approach does not require any assumptions regarding the correlation between the damage of structural components at various levels of ground motion intensity.

In the first step of the methodology (Fig. 1) it is necessary to assemble information about: the location of the building, its geometry, material and modelling characteristics, the classification of structural and non-structural components into fragility and performance groups, the corresponding fragility and loss functions and the replacement cost of the building.

In the second step, the results of hazard analysis at the location of the building have to be obtained from Probabilistic Seismic Hazard Analysis (PSHA). In addition to the seismic hazard curve, an adequate set of ground motions has to be selected. In order to consider the ground-motion randomness n_a ground motions were selected from the PEER Ground Motion Database [11] according to the procedure proposed by Jayaram et al. [12].

In the third step, the structural analysis is performed. In this step it is possible to explicitly incorporate the effect of epistemic uncertainties by a set of n_m structural models, whose modelling and material parameters are sampled from the statistical distributions with one of the sampling methods (e.g. Latin Hypercube Sampling technique). In this paper, the motivation was to show the effect of material quality on the seismic risk of the buildings, hence the explicit consideration of epistemic uncertainties was omitted in the examples, but due to completion they are included in the short presentation of the methodology (Fig. 1). Their effect on the results can be studied elsewhere [13].

The single-degree-of-freedom (SDOF) models are then defined for each of n_m structural models. The pushover curves are idealized by a simple trilinear force-displacement relationship, followed by a simple transformation from a multi-degree-of-freedom (MDOF) model to a SDOF model [14]. Finally, the incremental dynamic analysis is performed on the equivalent SDOF models [8]. The results are $n_a \cdot n_m$ SDOF-IDA curves where the engineering demand parameters (edp) for multiple intensity levels (im) are obtained until the seismic intensity im_C, which causes dynamic instability of the building. By combining the seismic demand from incremental dynamic analysis and the damage analysis based on the results of pushover analysis, it is possible to estimate the conditional probability of building's collapse $P(C|IM)$, the collapse fragility.

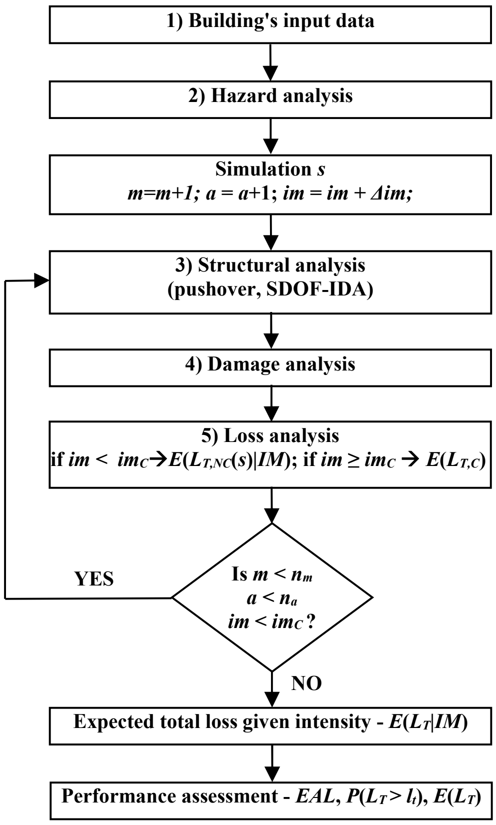

Fig. 1 – Overview of the methodology for seismic risk assessment

In addition, based on the values of the EDPs for all the structural and non-structural components in each simulation (steps 4 and 5) and by knowing the relationship between damage and engineering demand parameters (fragility functions) and the relationship between loss and damage (loss functions), the expected loss in each component j given EDP – $E(L_j|EDP_j(im))$ can be calculated:

$$E\left(L_j \mid EDP_j\left(im\right)\right) = \sum_{all\ DS_j} E\left(L_j \mid DS_j\right) \cdot p\left(DS_j \mid EDP_j\left(im\right)\right) \tag{1}$$

where the expected loss in component j for each damage state $E(L_j|DS_j)$ is weighed by the probability of its occurrence $p(DS_j|EDP_j(im))$ and summed over all possible local damage states. The total loss in the building for simulation s is simply the sum of expected losses over all components:

$$E\left(L_{T,NC}\left(s\right) \mid IM = im\right) \approx \sum_{all\ j} E\left(L_j \mid EDP_j\left(im\right)\right) \tag{2}$$

Note, that in the proposed methodology value of edp_j given im is obtained directly from structural analysis for each component. This sample is approximation of random variables $EDP_j(im)$ (Eq. (1)) and its size is $n_a \cdot n_m$. Index NC stands for the non-collapse case, which occurs in each simulation for the intensity levels $im < im_C$. However, for intensities higher than im_C, the expected total loss of the building is equal to its replacement cost including the cost of demolition $E(L_{T,C})$. For each intensity level im, the size of the sample values of $E(L_T(s)|IM)$ is also equal to $n_a \cdot n_m$. A mean value of $E(L_T(s)|IM)$ represents the expected total loss given intensity $E(L_T|IM)$, which is often termed the vulnerability curve. The expected annual loss (EAL) can be obtained by convolving the mean annual frequency of exceeding the ground motion intensity λ_{IM} and the expected total loss given intensity:

$$EAL = \sum_{all\ \mathrm{im}} E\left(L_T \mid IM\right) \cdot \left| \frac{\Delta\lambda_{IM}(im)}{\Delta IM} \right| \Delta IM \tag{3}$$

This performance measure is very important for the investors, owners and other stakeholders, since they can compare the expected annual loss to the insurance premiums or annual revenues for financial planning. By analogy to Eq. (3), the mean annual frequency of exceeding a certain total loss $\lambda(L_T > l_t)$ (loss hazard curve) can be computed by integrating the conditional probability of exceeding a certain loss given intensity $P(L_T > l_t|IM)$ over all possible levels of ground motion:

$$\lambda(L_T > l_t) = \sum_{all\ \mathrm{im}} P\left(L_T > l_t \mid IM\right) \cdot \left| \frac{\Delta\lambda_{IM}(im)}{\Delta IM} \right| \Delta IM \tag{4}$$

where $P(L_T > l_t|IM)$ is estimated from simulations.

3. Case study: two three-storey masonry buildings

3.1 Building's input data

The methodology is demonstrated by means of examples of three-storey unreinforced masonry buildings which have the same geometry but different quality of masonry. The plan and the elevation of the buildings are presented in Fig. 2. The buildings are symmetric around the Y axis. They have 5.6 % and 5.3 % of shear walls in the X and Y direction, respectively. The wall thickness is 0.3 m and the storey height of all storeys is 3.2 m. Concrete slabs with thickness of 0.18 m are considered as rigid diaphragms. It is assumed, that the buildings are located in Ljubljana (Slovenia) on the soil type B (Eurocode's terminology). One of the buildings is assumed to be built from modern European masonry of hollow clay bricks and the other represents traditional European masonry made from solid bricks.

The equivalent frame models of the buildings were made by using the research version of the program Tremuri [15], which is specialized for seismic analysis and performance assessment of masonry structures. The nonlinear model consisted of planar frames, which are connected at the corners and intersections of walls. Each wall of the building was divided into piers and spandrels, where the non-linear response is simulated in plastic hinges. The main advantage of such macroelement model is the capability of representing the shear sliding and flexural failure mechanisms with toe crushing and their evolution, controlling the strength and stiffness deterioration. Note, that the global behaviour was governed only by in-plane capacity of the walls, since out-of-plane collapse was not considered. Another important aspect of the mathematical model is the definition of ultimate drifts. The macroelement's lateral stiffness and strength were set to zero, if the drifts of the structural components exceeded the ultimate drifts.

The impact of epistemic uncertainty was not considered in this case, hence only deterministic model of each building was defined ($n_m = 1$). In the Table 1, 7 modelling parameters are presented, which were used for both material types. The information regarding the modelling parameters was mainly adopted from the literature, which is summarized in [10]. The only exceptions were the ultimate drifts in shear and flexure, which were determined based on the database of the experimental results.

The dead load of the first two storeys amounted to 6.2 kN/m^2, and to 6.3 kN/m^2 on the flat walkable roof. Since the building was assumed to be an office building, the live loads for the floors, balconies and staircases were 3 kN/m^2, 2.5 kN/m^2 and 2 kN/m^2, respectively.

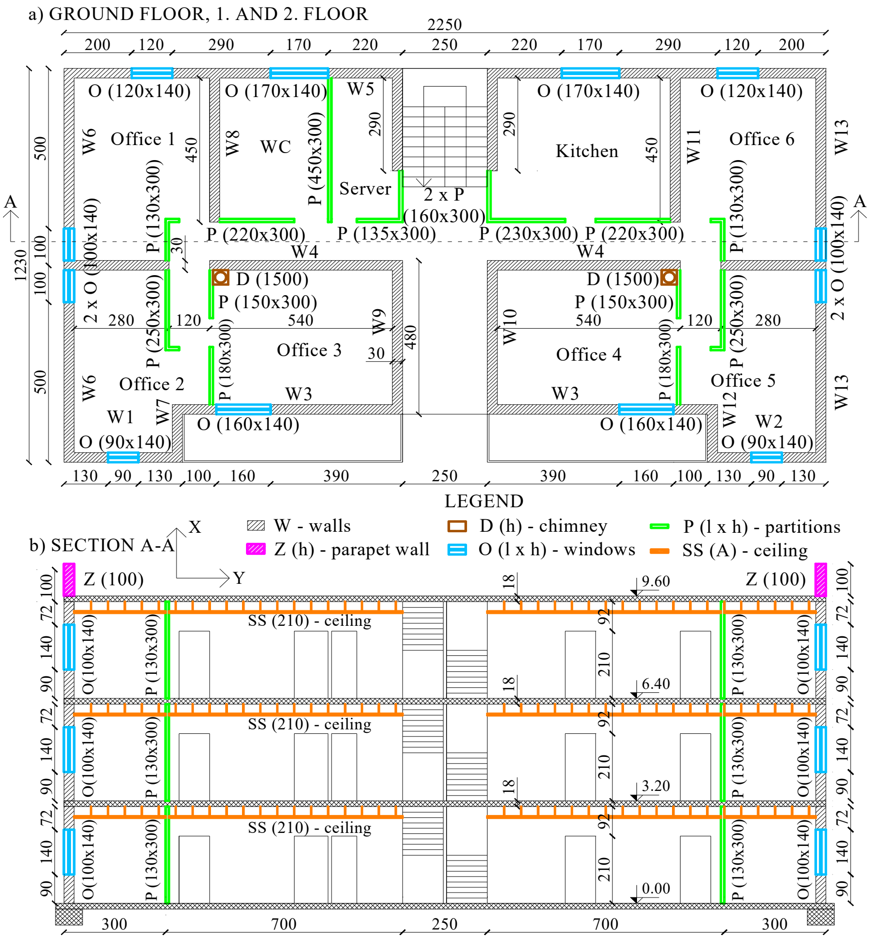

Fig. 2 – (a) The typical plan and (b) the elevation of buildings made from modern hollow clay bricks and traditional solid bricks. Presented are the structural and non-structural components.

Table 1 – The expected material parameters of buildings made from modern hollow clay bricks and traditional solid bricks.

Parameter	Modern masonry (hollow blocks)	Traditional masonry (solid bricks)
Specific weight γ (kN/m^3)	14	16
Comp. strength f_m (MPa)	5	2.5
Shear strength f_{v0} (MPa)	0.20	0.10
Elastic modulus E (MPa)	5000	1000
Shear modulus G (MPa)	500	250
Ultimate shear drift δ_s (%)	0.41	0.41
Ultimate flexural drift δ_f (%)	0.72	0.72

Additionally, the non-structural components were also considered in the analysis. They were categorized into fragility groups (the same fragility function) and performance groups (a logical group of components with similar performance). The description and the quantities for each of these groups are shown in the Table 2 together with the corresponding parameters of fragility functions (median value and *CoV* of the *EDP*) and loss functions (expected cost of repair compared to the cost of new component per unit) for each damage state. Finally, the cost for building's replacement including demolition was estimated to be 590000 € based on the Slovenian cost databases.

Table 2 – The database of fragility and performance groups used in this study. The median and coefficient of variation of the corresponding EDP define the fragility functions and the expected cost of repair compared to the cost of new element define the loss functions. Fragility functions are based on lognormal distribution.

Fragility and performance groups						Fragility functions				Loss functions	
	Components	**Unit**	**Floors**	**Quantity X**	**Quantity Y**	***DS***	***EDP***	$\tilde{X}$	***CoV***	**New unit cost (€)**	***E*(*L'*|*DS*)**
Structural	Masonry walls - shear failure	m^2	1, 2, 3	166	159	*DS1*	*IDR* (%)	0.11	0.26	101	0.21
						DS2		0.29	0.47	101	0.86
						DS3		0.41	0.57	101	1.21
	Masonry walls - flexural failure					*DS1*	*IDR* (%)	0.05	0.50	101	0.21
						DS2		0.33	0.52	101	0.86
						DS3		0.72	0.47	101	1.21
Non-structural	Partition walls	m^2	1, 2, 3	26	66	*DS1*	*IDR* (%)	0.21	0.60	37	0.30
						DS2		0.71	0.45	37	0.60
						DS3		1.20	0.45	37	1.20
	Windows	# of windows (1,4 m × 1,4 m)	1, 2, 3	7.7	2.9	*DS1*	*IDR* (%)	1.60	0.29	560	0.10
						DS2		3.20	0.29	560	0.60
						DS3		3.60	0.27	560	1.20
	Masonry parapet	m^2	3	25	45	*DS1*	*PFA* (g)	0.20	0.60	78	0.60
						DS2		0.40	0.60	78	1.20
	Masonry chimney	m	1	30		*DS1*	*PFA* (g)	0.35	0.60	150	1.20
						DS2		0.50	0.60	150	1.20
	Suspended ceiling	m^2	1, 2, 3	210		*DS1*	*PFA* (g)	0.27	0.40	22.5	0.12
						DS2		0.65	0.50	22.5	0.36
						DS3		1.28	0.55	22.5	1.20
	Server and computers	/ floor	1, 2, 3	12000		*DS1*	*PFA* (g)	1.00	0.50	1000	1.00
	Generic drift sensitive components	/ floor	1, 2, 3	20000		*DS1*	*IDR* (%)	0.40	0.50	1000	0.25
						DS2		0.80	0.50	1000	0.10
						DS3		2.50	0.50	1000	0.60
						DS4		5.00	0.50	1000	1.20
	Generic acceleration sensitive components	/ floor	1, 2, 3	20000		*DS1*	*PFA* (g)	0.25	0.60	1000	0.02
						DS2		0.50	0.60	1000	0.12
						DS3		1.00	0.60	1000	0.36
						DS4		2.00	0.60	1000	1.20

3.2 Hazard and structural analysis

The hazard curve (Fig. 3a) was obtained from previous study (Brozovič and Dolšek, 2013). The ground motions were selected to match the Eurocode's spectrum (Fig. 3b), which provides conservative estimates of the fragility functions at the level of the structure and losses. For the ground motion selection, the soil type B was assumed with consideration of the interval of the magnitudes ($5.5 \leq M \leq 7.5$), the source-to-site distances (5 km $\leq r \leq 50$ km) and scale factor ($sf \leq 3$). The peak ground acceleration was selected for the intensity measure.

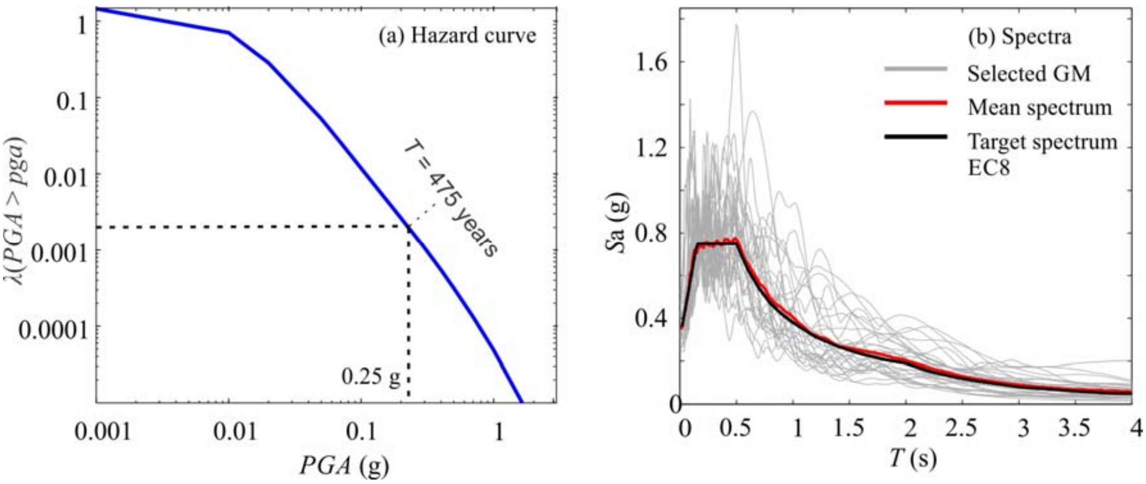

Figure 3 – (a) The hazard curve and (b) the elastic acceleration spectra of the 30 selected ground motions and target spectrum from Eurocode 8 for soil type B.

In Fig. 4 the results of structural analysis are summarized for the models of the building made from modern and traditional masonry. Firstly, the pushover analysis is performed, and then the pushover curve is idealized with a simple tri-linear force-displacement relationship and transformed from MDOF to SDOF model (Figs. 4a and 4b). Three global damage states were defined based on the pushover curve: minor damage at the 70 % of the maximum base shear F_{max} (DS1), medium damage at the maximum base shear F_{max} (DS2), where there is enough damage in the elements, that the building's resistance starts to decrease and the near collapse damage state with severe damage (DS3), where the base shear decreases below 80 % of F_{max}. Note the sudden strength deterioration in the pushover curve that occurred due to the formation of plastic mechanism in the first storey, where multiple walls failed at approximately the same displacement. In the case of the modern building, the ratio of the maximum base shear F_{max} and its weight W is 0.41 and the deformation capacity in terms of displacement at the top of the building is $d_{DS3} = 4.9$ cm. In the case of the building built from traditional masonry the ratio F_{max}/W is 0.31 (25 % decrease) and the deformation capacity is $d_{DS3} = 3.0$ cm (40 % decrease).

In Figs. 4c and 4d, the SDOF-IDA curves are shown for the set of 30 ground motions, which reflect the impact of ground-motion randomness, including the 16th, 50th and 84th percentile curves. Additional damage state of collapse (DS4) is defined in the decreasing part of the idealized pushover envelope, where a very small increment in acceleration results in a very large increase of the displacement, hence the building collapses. The seismic intensities causing damage states DS1-DS4 vary quite significantly although the structure had very low vibration period. For models of buildings from both types of masonry, a sample of 30 intensities, which cause the collapse of the building, was estimated. The median collapse capacity $pga_{50,DS4}$ and the corresponding dispersion β_{DS4} amounted to 0.58 g and 0.18 in the case of modern building and to 0.31 g (47 % decrease) and 0.19 in the case of traditional building, respectively.

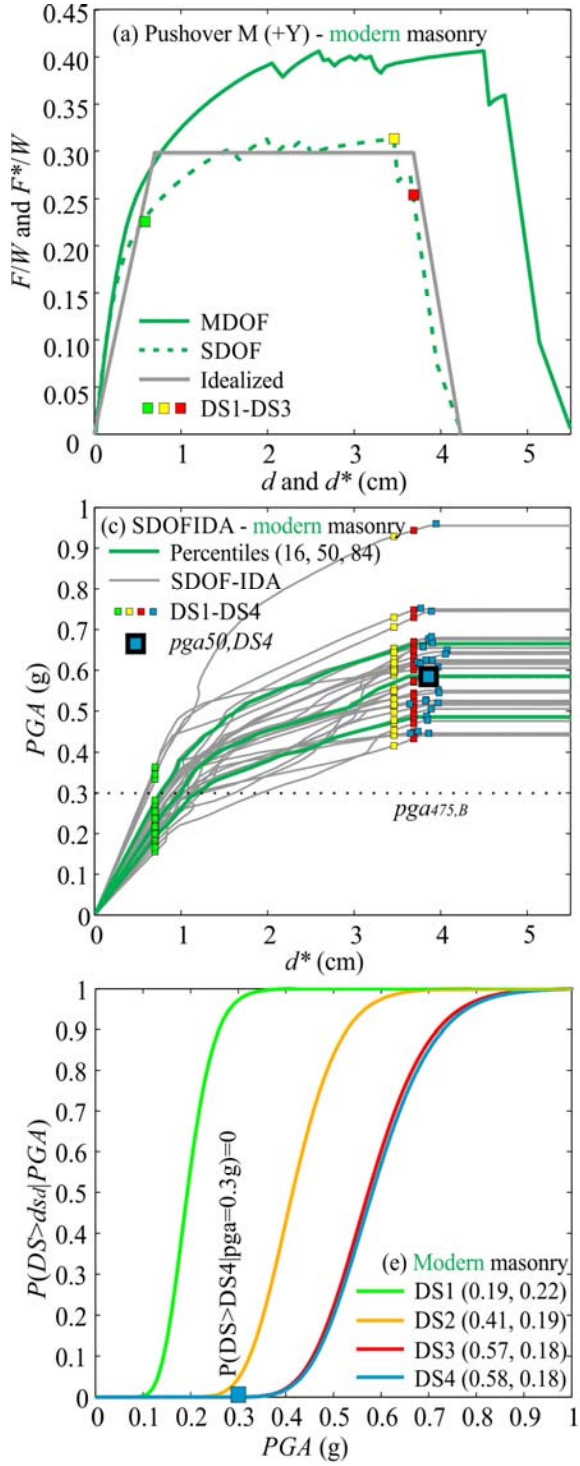

Figure 4 – The results of the structural and damage analysis for models of building built from modern and traditional masonry and comparison of: (a, b) the pushover curves for MDOF and SDOF model and idealized force-displacement relationship; (c, d) the SDOF-IDA curves for 30 ground motions and (e, f) the building's collapse fragility.

3.3 Damage and loss analysis

The fragility curves, which are defined as the conditional probability of damage exceeding a specified damage state given intensity $P(DS > ds_d|IM)$ are shown in Figs. 4e and 4f for both buildings including the corresponding median value of pga_{50} and dispersion β. Note that the probability of collapse for modern building in the case of the design earthquake with $pga = 0.30$ g is negligible. However, the conditional probability of collapse in the case of the same earthquake for traditional building is 41 %.

If the product of conditional probability of exceeding a certain damage state and the probability of occurrence of an earthquake with a certain *pga* are integrated over all possible values of *pga* (Eq. (4)), we obtain the probability that the building will experience a certain damage state. The probabilities that both buildings will experience a certain damage state DS1-DS4 in 1 or 50 years are shown in the Table 3. Quite high probability of experiencing minor damage due to earthquakes exist for both buildings, almost 16 % in case of modern building and almost 22 % in case of a building made from traditional masonry. There is also 1.2 % probability of collapse in 50 years for the modern building, however in case of traditional building, the risk of collapse is increased by a factor 4.5 to 5.3 %.

Table 3 – The probability that the building, built from modern and traditional masonry, will experience a certain damage state DS1-DS4 in 1 or 50 years.

		DS1	DS2	DS3	DS4
Modern masonry	$P(DS > ds_d \mid 1 \text{ year})$ (%)	0.34	0.06	0.024	0.023
	$P(DS > ds_d \mid 50 \text{ years})$ (%)	15.8	2.8	1.2	1.2
Traditional masonry	$P(DS > ds_d \mid 1 \text{ year})$ (%)	0.48	0.14	0.11	0.11
	$P(DS > ds_d \mid 50 \text{ years})$ (%)	21.8	7.0	5.6	5.3

The expected loss at the component level and the total loss for the non-collapse cases given the intensity were estimated according to Eqs. (1) and (2), respectively. At each intensity level *im* the top displacement for the MDOF model was obtained by transformation of the corresponding displacement from the SDOF-IDA curve. The so-determined top displacement was used to estimate the engineering demand parameters from pushover analysis. For example, the expected losses for non-collapse cases given the intensity are presented for different fragility groups (Figs. 5a and 5b). The contribution of the non-structural components is significant. In the case of modern masonry, they contribute more than 65 % to the total loss. In case of weaker traditional masonry, the damage and losses from structural components are higher, however the non-structural components contribute almost 50 % to the total loss. Note, that negligible losses were observed for windows. The main reason for this is the assumed fragility function (Table 2), because masonry buildings typically don't experience very high interstorey drifts and the windows remain undamaged in the simulations. In the reality, this is of course not the case, hence the fragility functions for some of the non-structural components should be investigated thoroughly in the future research, or the correlations between damage of various components should be considered (i.e. windows are damaged if the adjacent masonry wall fails). Although the proposed methodology enables explicit consideration of such correlations, they were not considered in this study due to simplicity.

In Figs. 5c and 5d the expected total loss given intensity $E(L_T|IM)$ is presented, considering also the simulations where the collapse of the building occurs. Note that the collapse cases occurred even at $pga = 0.25$ g. Therefore the total loss due to collapse start to dominate quickly, because the ratio between replacement cost of the building $E(L_{T,C})$ and the expected losses given no collapse $E(L_{T,NC}|IM)$ is very high in this case. The collapse fragility of the building and the ratio $E(L_{T,C})/E(L_{T,NC}|IM)$ were also the most influential parameters for the $E(L_T|IM)$ [10]. At higher intensities, where the probability of collapse becomes 1, the expected total loss is equal to replacement cost of the building ($E(L_{T,C})$).

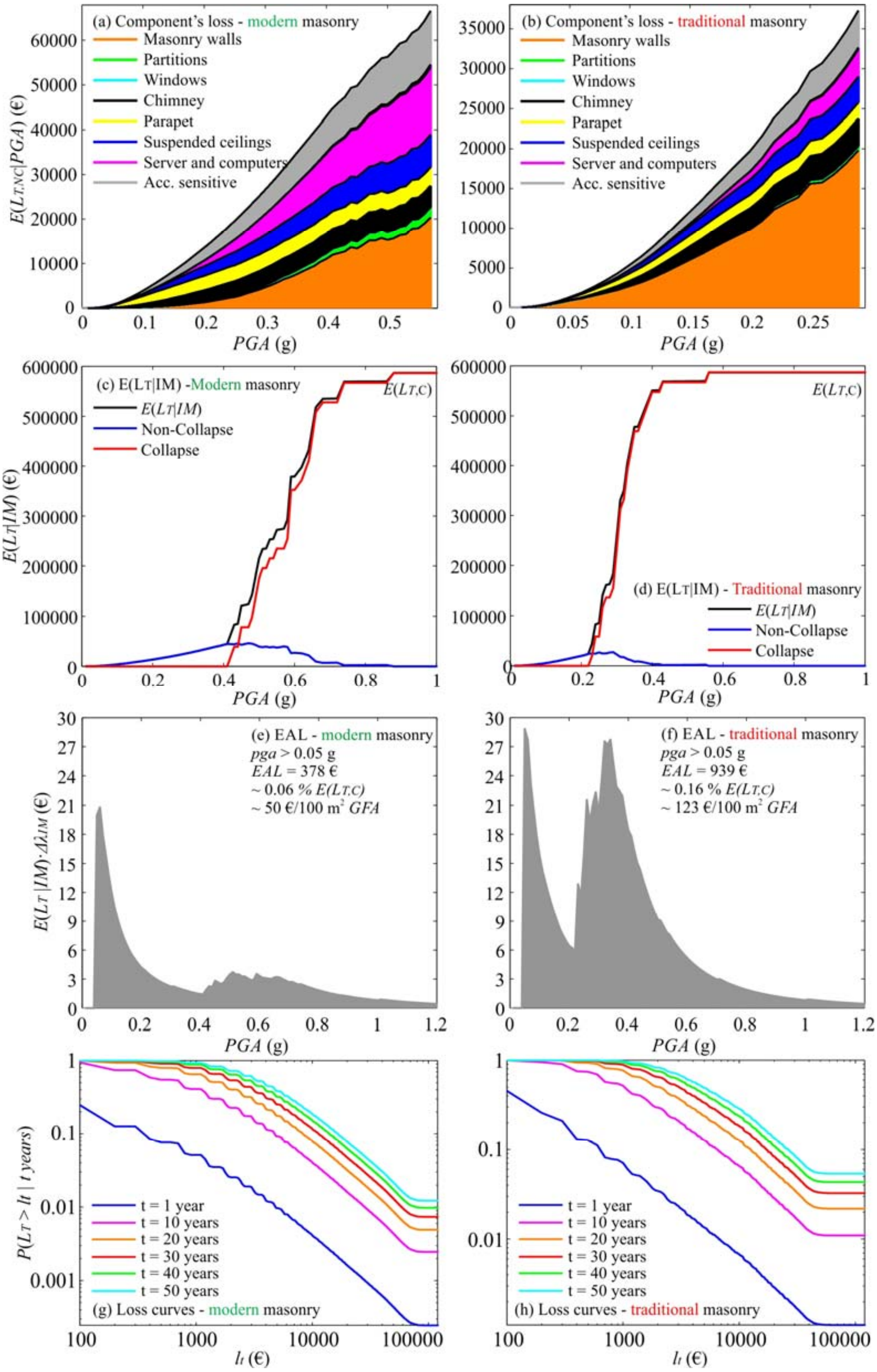

Figure 5 – The results of the loss analysis for models of buildings built from modern and traditional masonry and comparison of: (a, b) the contribution of the considered fragility groups to the $E(L_{T,NC}|IM)$, (c, d) the disaggregation of the expected losses given intensity, (e, f) the disaggregation of *EAL* by intensity measure for *pga* > 0.05 g and (g, h) the loss curves for multiple timeframes.

It is also interesting to investigate the expected annual loss *EAL* (Eq. (3)). The *EAL* was estimated to 378 € (~0.06 % replacement cost or 50 € per 100 m^2 of gross floor area) for the modern building and to 939 € (~0.16 % of the replacement cost or 123 € per 100 m^2 of gross floor area) for the traditional building. In Figs. 5e and 5f, the disaggregation of EAL by intensities is presented. Note the two peaks in the graph, which represent the large contribution of minor but frequent earthquakes to the expected annual loss and also the contribution of stronger earthquakes, which are able to collapse the building, however they aren't very frequent. Extremely strong earthquakes with $pga > 1$ g don't contribute a lot to the *EAL*, since they are considered extremely rare in Slovenia and thus they have very small probabilities of occurrence. Note also, that the contribution of very frequent earthquake with $pga < 0.05$ g is neglected, since this earthquake typically don't cause noticeable damage and the owners usually don't repair the buildings after such events.

Finally, the results of the loss assessment are presented in terms of loss curves (Eq. (4), Figs. 5g and 5h). The investors can get very interesting information from these curves. For example, the expected loss which is exceeded with 10 % probability in 50 years was estimated to 17000 € and 25000 € for modern and traditional building, respectively. Another way to communicate to the stakeholders in terms of losses is to estimate the probability that the loss will exceed a certain value. For example, there are 0.44 % and 1.11 % probability that the losses will exceed 50000 € in 10 years in case of modern and traditional building.

4. Conclusions

In this study, a methodology was presented for seismic risk assessment of buildings in terms of various measures including probability of collapse and the expected losses due to earthquakes. Loss estimation is often performed under the assumption of uncorrelated damage in components, which can potentially lead to biased results, however the presented methodology enables explicit consideration of correlation between damage in various components.

The combination of pushover analysis, which is performed for the model of entire structure, and incremental dynamic analysis for the SDOF model was applied to realistic structures in order to study the effect of material quality on the results. In the case of the modern building, the ratio F_{max}/W and displacement capacity of the building in terms of displacement at the top was 30 % and 60 % higher, than in the case of traditional masonry building, respectively. The median collapse capacity and the corresponding dispersion of collapse capacity of the building expressed in terms of *pga* amounted to 0.58 g and 0.18 in the case of modern building and to 0.31 g and 0.19 in the case of traditional building. Consequently, 4.5 times higher probability of collapse was estimated for the traditional building (4.5 % in 50 years) compared to the modern building (1.2 %).

In terms of losses, the contribution of the non-structural components is significant, since they contributed more than 50 % to the total loss for both buildings. The *EAL* of the modern building was estimated to 50 € per 100 m^2 of gross floor area for the modern building and to 123 € per 100 m^2 gross floor area for the traditional building. There is also 0.44 % and 1.11 % probability that the losses will exceed 50000 € in 10 years in case of modern and traditional building, respectively. All the results indicate much higher seismic risk of traditional masonry buildings compared to the modern buildings.

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