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P.B. Lourenço, P. Roca, C. Modena, S. Agrawal (Eds.)

Approach to Assess the Seismic Risk of Historical Churches

M. Urban
International Graduate College 802 "Risk Management on the Built Environment", TU Braunschweig, Braunschweig, Germany

S. Sperbeck
International Graduate College 802 "Risk Management on the Built Environment", TU Braunschweig, Braunschweig, Germany

U. Peil
Institute of Steel Structures, TU Braunschweig, Braunschweig, Germany

ABSTRACT: The task of analysing the seismic risk inherent in monumental structures, which may be expressed as structural damage, loss of live or loss of cultural, historical or social values, is still a task not sufficiently solved. Risk is a measure which should not be described in deterministic, but probabilistic terms. It is thus the task of engineers to provide at least information about the reliability of deterministic results and even more important about the possible scatter of the considered output. Within this paper, the impact of the variability of ground motions and material parameters is analysed. Due-to space restrictions the presentation of results will be focused on a typical vault and a wall found in many churches.

1 PROBLEM STATEMENT AND SOLUTION APPROACH

After earthquakes a large part of the damaged structures belongs to historical or monumental buildings, even in regions with only a small seismic hazard like Germany as may be seen in the historic sources quoted by Amstein (2005). This is why a lot of present research is focused on these types of buildings (Lagomarsino et al. 2002, Lourenco and Oliveira 2005). Typical examples are historical churches which exhibit a high vulnerability due to their tall structure, vaulted components and the masonry material commonly used. Due to the large number of these structures, the possible resulting losses of human life or cultural values are often extremely tragic. The best known example is certainly the partial collapse of the Basilica Superiore di San Francesco in Assisi during the 1997 Umbria-Marche earthquake, but numerous other examples exist. It is thus a critical task to predict possible damages and their probability of occurrence.

Within this field, the macroelement approach has established itself in various applications (Doglioni et al. 1994, Augustin et al. 2002, Lagomarsino et al. 2002). Despite increasing computer speed and more sophisticated material models, this still seems to be the most practical and feasible concept to assess the vulnerability and resulting risk of larger structures. So far, the seismic capacity of these macroelements was assessed only by means of nonlinear static procedures using deterministic parameters in nearly all cases. Augustin et al. (2002) were the first to include a probabilistic study. They assigned probability density functions to the seismic coefficients based on the number and significance of the geometrical and material data for given earthquake intensities.

The purpose of this study is to offer a more detailed insight into the influence of load and material uncertainties with respect to historical masonry structures subjected to earthquakes. Throughout this study it is important to realize, that this work is done in the context of giving a prediction of the probability distribution of possible damages and their consequences and not to assess the structural safety itself. For further information on the risk process the interested reader is referred to Urban (2004). To reach this goal, a three step procedure is followed. At first, those earthquake parameters are determined which are best correlated with the damage occurring in a structure. The focus lies on the effects of PGA and duration. The influence of duration is still not fully understood (Bommer et al. 2004). Next, PGA and duration occurrence rates are determined. Finally, the results are implemented into nonlinear dynamic Monte Carlo simulations of a wall and a vault.

2 HAZARD ASSESSMENT

2.1 Correlation of strong motion intensity parameters and structural damage

Several intensity parameters for the description of earthquakes exist. A very good overview is given by Bommer et al. (2004), who also performed a study on the correlation of intensity parameters with structural damage. In order to assess the strong motions effects, a sample study was performed of which short results shall be presented. An artificial earthquake based on the summation of harmonic components with randomly chosen phase angles was generated and fitted to a response spectrum. This accelerogram will be referred to as "root" earthquake. The length of this root accelerogram was set to eleven seconds with a linearly increasing intensity function during the first second and a decreasing one in the last second. Now a set of 100 earthquakes was created by multiplying the amplitudes of the root earthquake by factors 0.5 to 5.0 in steps of 0.5 and changing the length of the accelerogram from two seconds to 11 seconds in steps of one second. The following Fig. 1 shows the root accelerogram and the Pseudovelocity spectra depending on duration and PGA of the record.

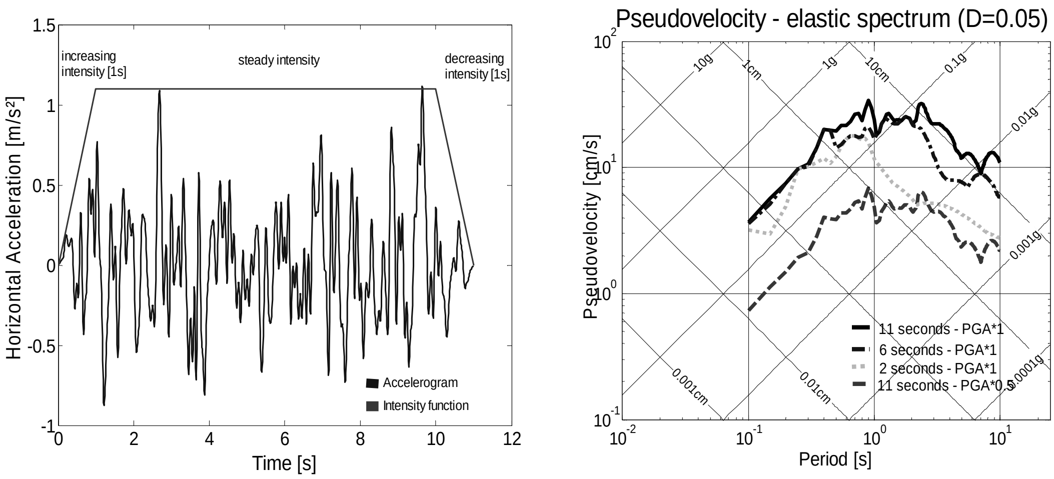

Figure 1 : The root accelerogram, its intensity curve and the corresponding pseudovelocity spectra.

Although this approach does not reflect the true variability of ground motions, it is especially helpful since the impact of duration and PGA can be assessed rather easily, for the influence of other intensity parameters is not altering in the same order of magnitude as it would if natural accelerograms are used. The set of 100 earthquakes was then applied to a finite element model of a wall with changing height, i.e. different first eigenfrequencies ranging from 1 Hz to 10 Hz. The material model used in this study was developed by Gambarotta and Lagomarsino (1996). It is able to describe the post-peak behaviour of the masonry material and the dynamic behaviour by dissipation through frictional mechanisms. It is a damage model in which the inelastic strains are described by means of two internal damage variables that describe the damage evolution in the bricks and in the mortar. Five structural damage parameters were used. At first the two internal damage variables describing the loss of strength in mortar and brick are considered. Additionally the vertical plastic strains and shearing plastic strains as well as the horizontal displacement at the top of the wall were regarded as damage indicators. For the two damage variables and the plastic strains the maximum value occurring through the time history was taken. For those four parameters the accumulation of all values over time and structure was considered as well. Some of the results of this study are presented in terms of rank order correlation coefficients in Table 1.

Table 1 : Correlation coefficients for correlation of damage and intensity parameters.

	ux	damage mortar		damage stone		vertical strain		horizontal strain	
Parameter		max	sum	max	sum	max	sum	max	sum
10 Hz									
PGA*	0.95	0.97	0.96	0.98	0.97	0.98	0.76	0.76	0.94
AI*	0.89	0.88	0.88	0.89	0.89	0.90	0.67	0.90	0.77
I_D*	0.17	0.14	0.14	0.12	0.14	0.15	0.05	0.20	0.11
E_D*	0.83	0.81	0.81	0.81	0.82	0.83	0.61	0.84	0.71
$S_a(f)$*	0.95	0.95	0.86	0.95	0.97	0.97	0.75	0.93	0.83
VSI*	0.91	0.89	0.89	0.90	0.90	0.92	0.69	0.92	0.77
NC*	0.15	0.11	0.11	0.09	0.11	0.12	0.01	0.17	0.08
$D_{u\,0.05}$*	0.73	0.69	0.69	0.71	0.72	0.72	0.50	0.73	0.59
$D_{b\,0.05}$*	0.46	0.41	0.41	0.42	0.45	0.44	0.27	0.46	0.35
$D_{s\,95}$*	0.16	0.12	0.12	0.10	0.12	0.13	0.03	0.19	0.08
5 Hz									
PGA	0.94	0.92	0.93	0.95	0.96	0.95	0.88	0.93	0.88
AI	0.94	0.89	0.92	0.92	0.94	0.93	0.72	0.92	0.80
I_D	0.27	0.18	0.24	0.21	0.25	0.22	0.00	0.24	0.09
E_D	0.89	0.82	0.86	0.86	0.89	0.87	0.63	0.87	0.74
$S_a(f)$	0.94	0.94	0.91	0.95	0.97	0.96	0.81	0.93	0.86
VSI	0.94	0.90	0.93	0.93	0.95	0.94	0.74	0.94	0.81
NC	0.25	0.15	0.21	0.19	0.23	0.19	0.02	0.21	0.05
$D_{u\,0.05}$	0.79	0.76	0.75	0.76	0.80	0.79	0.52	0.77	0.61
$D_{b\,0.05}$	0.55	0.48	0.48	0.49	0.56	0.50	0.26	0.50	0.34
$D_{s\,95}$	0.26	0.17	0.23	0.20	0.24	0.20	0.02	0.22	0.07
2 Hz									
PGA	0.83	0.74	0.84	0.85	0.88	0.82	0.65	0.78	0.77
AI	0.94	0.87	0.96	0.95	0.97	0.95	0.58	0.88	0.83
I_D	0.42	0.37	0.40	0.36	0.37	0.41	0.02	0.39	0.25
E_D	0.94	0.87	0.95	0.94	0.95	0.95	0.56	0.88	0.82
$S_a(f)$	0.92	0.88	0.93	0.94	0.96	0.93	0.62	0.87	0.80
VSI	0.94	0.87	0.97	0.96	0.98	0.95	0.59	0.89	0.84
NC	0.39	0.34	0.37	0.33	0.33	0.38	0.01	0.36	0.25
$D_{u\,0.05}$	0.86	0.87	0.90	0.89	0.88	0.91	0.47	0.84	0.73
$D_{b\,0.05}$	0.64	0.70	0.66	0.65	0.64	0.69	0.25	0.65	0.51
$D_{s\,95}$	0.42	0.35	0.39	0.35	0.36	0.40	0.00	0.37	0.26
1 Hz									
PGA	0.76	0.85	0.76	0.85	0.93	0.86	0.65	0.84	0.83
AI	0.93	0.93	0.81	0.92	0.96	0.93	0.61	0.97	0.82
I_D	0.49	0.35	0.34	0.33	0.27	0.35	0.66	0.47	0.24
E_D	0.95	0.91	0.81	0.89	0.91	0.91	0.61	0.96	0.80
$S_a(f)$	0.90	0.90	0.83	0.89	0.91	0.91	0.55	0.95	0.78
VSI	0.93	0.94	0.81	0.93	0.97	0.94	0.62	0.97	0.84
NC	0.47	0.32	0.32	0.30	0.24	0.32	0.07	0.43	0.21
$D_{u\,0.05}$	0.89	0.81	0.67	0.78	0.85	0.80	0.49	0.89	0.68
$D_{b\,0.05}$	0.70	0.56	0.48	0.53	0.58	0.55	0.27	0.72	0.47
$D_{s\,95}$	0.47	0.31	0.31	0.29	0.22	0.31	0.06	0.43	0.19
0.5 Hz									
PGA	0.57	0.71	0.71	0.73	0.34	0.79	0.45	0.84	0.61
AI	0.81	0.86	0.86	0.87	0.52	0.91	0.32	0.92	0.68
I_D	0.59	0.40	0.45	0.39	0.38	0.38	0.08	0.31	0.12
E_D	0.86	0.87	0.87	0.87	0.58	0.91	0.26	0.89	0.68
$S_a(f)$	0.78	0.90	0.92	0.92	0.66	0.94	0.32	0.83	0.68
VSI	0.81	0.86	0.86	0.86	0.52	0.91	0.34	0.91	0.70
NC	0.57	0.39	0.42	0.37	0.39	0.36	0.14	0.28	0.09
$D_{u\,0.05}$	0.85	0.78	0.75	0.78	0.45	0.80	0.12	0.87	0.62
$D_{b\,0.05}$	0.77	0.57	0.55	0.57	0.36	0.57	0.02	0.68	0.35
$D_{s\,95}$	0.58	0.41	0.44	0.39	0.42	0.38	0.11	0.29	0.11

PGA: Peak Ground Acceleration; AI: Arias Intensity; I_D: Damage Factor by Consenza and Manfredi (2000); E_D: Specific Energy Density; $S_a(f)$: Spectral Acceleration in the first eigenfrequency of the structure; VSI: Velocity Spectrum Intensity; NC: Number of zero crossings; $D_{U\,0.05}$: Uniform Duration, threshold of 0.05g; $D_{B\,0.05}$: Bracketed duration, threshold of 0.05g; $D_{S\,95}$: Significant duration between 5% and 95% of the released energy.

Quite often a high correlation is achieved, as it can be seen especially for the high frequencies. This is due-to the fact, that the rank order correlation coefficient was used. In some cases linear correlations differ significanty because of the nonlinear relationship between the two variables.

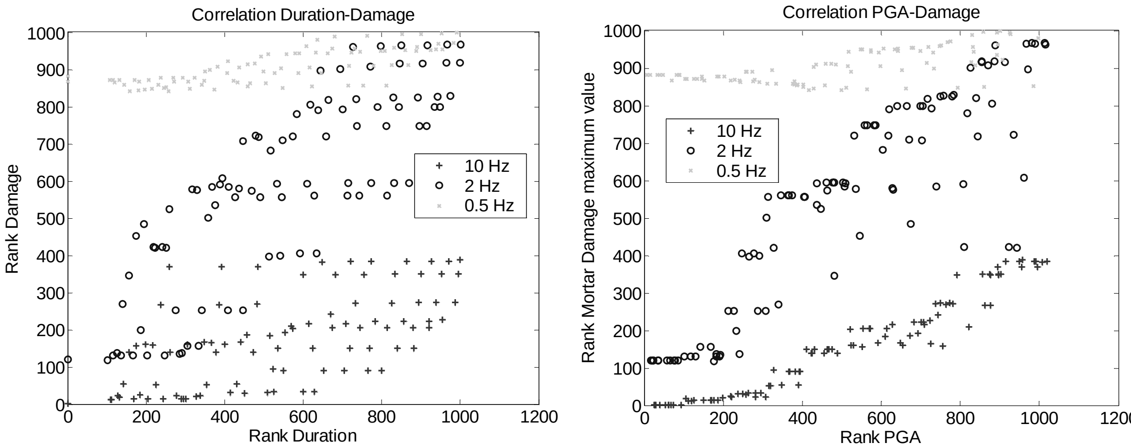

Figure 2 : Rank Order Correlations of Maximum Mortar Damage and Uniform Duration with 0.05g threshold for the full range of simulations.

Number of Zero crossing and the Damage Factor were not found to correlate well with the damage parameters observed. Specific Energy Density and Arias Intensity correlate most constantly over the considered frequencies. Of all duration measures, the uniform duration was evaluated to be best connected to the structural damage. It is especially well correlated within the frequency range of 0.5-5 Hz. The results also suggest that an important factor is the height/width ratio of the wall (cf. Fig. 2), which is because the occurring tensile and compressive stresses vary larger in higher structures. With the given results it can be concluded also, that for this task it is sufficient to provide estimates of duration and PGA in order to assess the probabilities of damage.

2.2 Probability of occurrence of intensity parameters

Occurrence rates or probabilities of exceedance may now be determined. They depend largely on the seismicity of a region, which means the distribution of magnitudes. The occurrence of magnitudes might either be described by an extreme value distribution type III for maxima as applied by Sanchez-Silva and Rackwitz (2004) or by the more common Gutenberg Richter approach. In Fig. 3 the distributions for both approaches are shown.

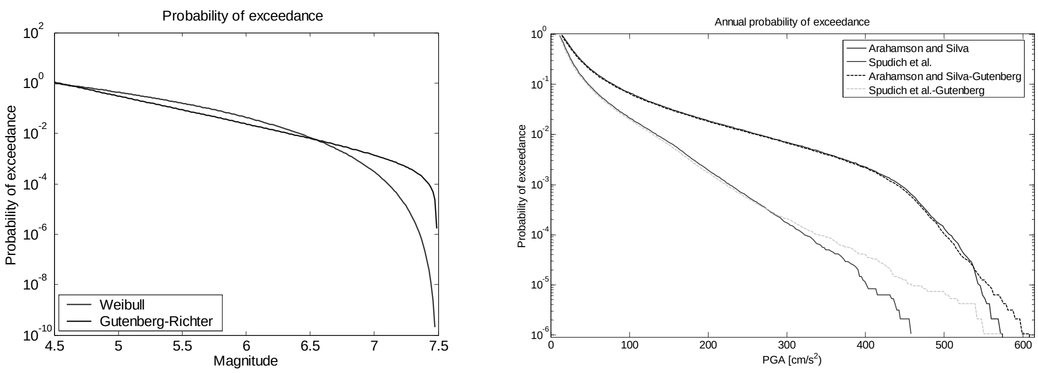

Figure 3 : Annual probabilities of exceedance for magnitudes (left) and PGA for two different attenuation functions (right).

Background data are the yearly occurrence rate of 1.12, the parameter u=1, w=4.5 for the extreme value distribution and the parameters a=5.0 and b=1.1 for the Gutenberg-Richter relationship. The advantage of the Weibull distribution is, that it is able to describe the rapid decrease of probability for magnitudes close to the maximum value. The local magnitudes considered fall within the interval of 4.5 and 7.5.

To assess the annual probabilities of exceedance of the PGA, the possible location of an earthquake was taken to be uniformly distributed over an area with a radius of 100km. With these parameters given the yearly occurrence rate may be determined by using attenuation functions which were proposed for example by Spudich et al. (1999), Abrahamson and Silva (1997), Sabetta and Pugliese (1996) or Ambraseys et al. (1996). For a more detailed overview of the results the interested reader is referred to Urban (2007).

2.3 Results

Five attenuation functions for the PGA and three for duration were considered. The main characteristics of the probability distributions after 1 million simulations for the PGA are shown in Table 2.

Table 2 : Main characteristics of the probability distribution of PGA

Attenuation function	Mean cm/s²	Standard Deviation cm/s²	Minimum cm/s²	Maximum cm/s²
Sabetta and Pugliese	16.11	14.22	6.05	517.57
Abrahamson and Silva	32.01	33.51	10.8	596.03
Ambraseys et al.	22.44	18.15	9.78	541.93
Joyner et al.	33.23	28.79	10.0	501.32
Spudich et al.	21.93	19.22	9.05	454.83

As can be seen in Fig. 3, the probability distribution for PGA – especially for the Spudich et al. (1999) attenuation – may be reasonably approximated by an exponential distribution. This assumption seems also justified, since mean and standard deviation as seen in Table 2 are values, which are very close. Thus for the subsequent calculations the PGA is assumed to be distributed exponentially. The duration was assumed to follow the dependence of the Sabetta and Pugliese (1997) attenuation function, whose annual probability of exceedance is roughly falling down in an exponential manner after a duration of 3 seconds.

3 VULNERABILITY DETERMINATION

3.1 Description of the building

The research task started with a church located close to the City of Aachen in Germany. The church, which was built in the 13th century, was damaged in an earthquake which occurred in 1992 (M_S=5.9, distance=39km) and again in an even smaller event (M_S=4.8, distance=20km) in 2002. This church is quite common in ground plan and is thus taken as an example for churches standing in regions with a higher seismicity as those described by the parameters in chapter 2. In this case, macroelements have been derived by analysing a linear elastic model at first.

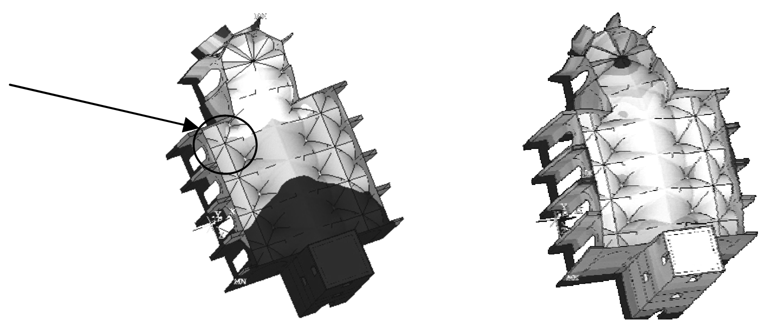

Figure 4 : 1st and 2nd Eigenmode of the Church, considered elements.

In this way it is possible to identify highest stressed parts and the location of the most endangered parts of the considered building. Also, this approach includes behavioural aspects instead of identifying macroelements merely on the basis of typology. Fig. 4 shows a modal analysis of the considered church. Shown are the total displacements for the first two modal shapes. As it can be seen, the structure may be easily subdivided into the components of abside, nave and tower. For space limitations and to underline the second major topic of this paper the results will be explained only for the vault with the largest differential displacements and its supporting wall. This vault may be found either on the right or left side of the nave closest to the abside (cf. arrow in Fig. 4). The considered wall will be the one supporting this vault in the direction of the abside.

3.2 *Input parameters*

To assess the reaction of these elements the material model of Gambarotta and Lagomarsino (1997) was used. The material properties were set after distributions measured Schueremans (2001) which are assumed to be representative for historic masonry. The parameters, their type of distribution and the main values are shown in Table 3.

With this information given, Monte Carlo Simulations were performed to evaluate the influence of the parameters, the sensitivity of input and the distributions of output parameters.

Table 3 : Parameters used and applied distributions.

Symbol	Variable	Distribution	Expected	Std. Deviation	Minimum	Maximum
μ	friction coefficient	lognormal	0.6	0.111	-	-
σ_{mr}	Tensile strength of mortar joints	lognormal	0.15 N/mm²	0.0525 N/mm²	-	-
τ_{mr}	Shear strength of mortar joints	lognormal	0.20 N/mm²	0.06 N/mm²	-	-
σ_{br}	Compressive strength of masonry	lognormal	3.5 N/mm²	0.665 N/mm²	-	-
τ_{br}	Shear strength of masonry	lognormal	1.5 N/mm²	0.45 N/mm²	-	-
η	Poisson ratio	lognormal	0.1	0.0125	-	-
ρ	Density	normal	2000 kg/m³	150 kg/m³	-	-
E	Young's Modulus	normal	2000 N/mm²	240 N/mm²	-	-
β_m	Softening mortar	triangular	0.8	-	0.3	1.0
β_b	Softening brick	triangular	0.4	-	0.3	1.0
c_{mt}	inelastic distortion mortar	uniform	1.0	-	0.5	1.5
c_{bt}	inelastic distortion brick	uniform	1.0	-	0.5	1.5
adamp	Rayleigh mass damping	uniform	0.62	-	0.3875	0.8215
bdamp	Rayleigh stiffness damping	uniform	0.0003	-	1.875E-04	0.0004

3.3 *Sensitivities and scatter for structural components*

Concerning the sensitivity of the input parameters results differ for wall elements and vaults. For the wall, the results are as it would be expected (cf. Fig. 5 right side). The most important parameters in the correct order are the horizontal acceleration (xskal), the density (dens) and the elastic modulus (emod) of the material. Most important means, that they have the largest influence on the damage parameters described in table 1. In second order, other material parameters also show some effect. They change for each considered variable, for the mortar damage σ_{mr} and τ_{mr} are dominating, whereas for brick damage – as shown in Fig. 5 – σ_{br} has the largest influence. For vaults, the matter is a different one. Interestingly, the horizontal acceleration is not dominant any more. Instead, material parameters govern the damage with special influence on the shear strength of the mortar joins (mshea), the softening behaviour and the compressive strength of the bricks (compm). Regarding the load factors, the vertical acceleration (yskal) and largest differential displacement at the piers (dis) exhibit the largest impact on the result in general.

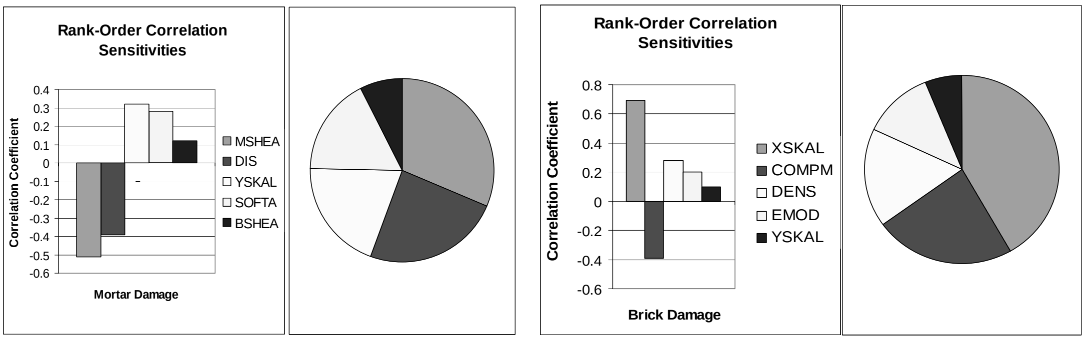

Figure 5 : Sensitivities for mortar damage in a vault (left) and sensitivities for brick damage in wall (right)

General remarks, which can be given about the type of distribution of the output parameters are that all type of stresses are distributed in a lognormal manner as it may be seen in Fig. 6. The distributions for the damage variables exhibit more an exponential type of distribution. In the applied material model the strains are linearly dependent on the stresses plus an inelastic contribution governed by the damage variables. Thus, for moderate events without high stress occurrence, the strains are lognormal distributed, whereas for higher ground accelerations, the inelastic contribution governs and strains are also exponentially distributed. Additional results were already described in Urban and Peil (2005). The distribution types of output parameters are similar for walls and vaults.

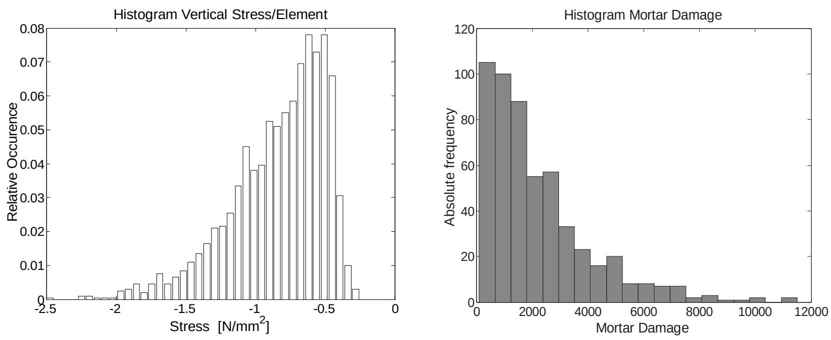

Figure 6 : Examples for distributions of stress in a wall and damage in a vault

The high influence of the horizontal acceleration on the scatter of the output parameters could imply that the distributions would follow different patterns, if only material parameters are varied and the ground motion stays the same. Thus, studies were also carried out with a constant ground motion and altering material variables. In this case, naturally, the range of results reduces with respect to the minimum and maximum values achieved. Still, the type of distribution stays the same.

4 CONCLUSIONS AND OUTLOOK

Within the risk assessment of historic structures several tasks have to be performed in order to be able to express the risk in probabilistic terms. Not everything could be explained in detail within this paper, but the authors will be happy to provide the reader with further information on request. This paper summarized distribution types for material properties and derived distributions for major ground motion intensity parameters. In a second step, the importance of load and material parameters for the structural assessment was assessed by means of nonlinear dynamic Monte Carlo Simulations using a suitable material model for masonry. Although the results presented here reflect the outcome of calculations with parameter input as presented in table 3, the outcome is transferable to other problems. Naturally, mean values and deviations will differ for each considered structure. Still, the general form of the distribution for the output parameters remains the same, so that with a small number of deterministic studies a suitable distribution can be assumed with the background of the data here. Also, in case of insufficient knowledge of the material parameters the scatter of deterministic studies can be easily assessed.

The last step of the risk assessment is to relate the calculation results to possible losses, which occur as a consequence of structural damage. Therefore, the result outcomes are connected to the damage degrees of the European Macroseismic Scale, by an approach described by Lang (2002) but extending it to vaults. With this done, we will be able to predict losses in dependence of the damage degree.

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REFERENCES

Abrahamson, N.A., Silva, W.J., 1997. *Empirical response spectral attenuation relations for shallow crustal earthquake*, Seismological Research Letters, Volume 68, pp. 94–127.

Ambraseys, N.N., Simpson, K.A., Bommer, J.J., 1996. *Prediction of horizontal response spectra in Europe*, Earthquake Engineering and Structural Dynamics, Volume 25, pp. 371–400.

Amstein, S., Lang, D. H., Schwarz, J., 2005. *Schütterwirkungen historischer Erdbeben und aktuelle Anwendungsgebiete für das Erdbebeningenieurwesen*. Bautechnik, 82, Heft 9, pp. 641-656.

Augusti, G., Ciampoli, M., Giovenale, P., 2002, Assessment of the seismic reliability of heritage buildings. In *Towards a History of Construction*, Becchi, A., (eds.), Birkhäuser, Basel, Switzerland.

Bommer, J. J., Magenes, G., Hancock, J., Penazzo, P.,2004, The Influence of Strong-Motion Duration on the Seismic Response of Masonry Structures, Bulletin of Earthquake Engineering, Volume 2-1 1.

Consenza, E., Manfredi, G. 2000, *Damage Indices and Damage Measures*, Progress in Structural Engineering and Materials, Volume 2, pp. 50-59.

Doglioni F., A. Moretti, A., Petrini,V., 1994, *Le Chiese e il terremoto*, Edizioni LLINT, Trieste, Italy.

Gambarotta, L., Lagomarsino, S. 1996. *Damage models for the seismic response of brick masonry shear walls. Part I and II*. Earthquake Engineering and Structural Mechanics, vol. 26, pg. 441-462.

Lang, K., 2002. *Seismic vulnerability of existing buildings*. Zürich: vdf, Hochschulverlag an der ETH.

Lagomarsino S., Podestà S., Resemini S., (2002). *Seismic response of historical churches*, Proc.12th European Conference on Earthquake Engineering, London.

Lourenço, P.B., Oliveira, D.V., 2005, *Seismic vulnerability of historical churches*, web (April 2006.): http://www.civil.uminho.pt/masonry/Publications/Update_Webpage/2005_PT-US%20Workshop2.pdf.

Sabetta, F., Pugliese, A., 1996. *Estimation of Response Spectra and Simulation of Nonstationary Earthquake Ground Motions*. Bulletin of the Seismological Society of America, Vol. 86, No.2, pp.337-352.

Sánchez-Silva, M., Rackwitz, R., 2004, *Socioeconomic Implications of Life Quality Index in Design of Optimum Structures to Withstand Earthquakes*, J. Struct. Engrg., Volume 130, Issue 6, pp. 969-977.

Schueremans, L. 2001. *Probabilistic evaluation of structural unreinforced masonry*. Faculteit Toegepaste Wetenschappen, Katholieke Universiteit Leuven.

Spudich, P., W. B. Joyner, A. G. Lindh, B. M. Margaris, D. M. Boore, and J. B. Fletcher, 1999, *SEA99 - A revised ground motion prediction relation for use in extensional tectonic regimes*, BSSA, v. 89, no. 5, p.1156-1170.

Urban, M., Peil, U., 2004. *Seismic risk assessment of monumental structures*, Disaster and society – from hazard assessment to risk reduction, Proceedings, Eds.: Mahlzahn, D., Plapp, T., Berlin, pp.145-154.

Urban, M., 2007, *Seismic risk assessment of historical structures,* PhD Thesis, TU Braunschweig, to be published in February 2007.

Urban, M., Peil, U., 2005: *The process of risk management and the influence of uncertainties in the risk assessment of historical structures*, Safety and Reliability of Engineering Systems and Structures, ICOSSAR '05, Proceedings, Eds.: Augusti, G., Schueller, G., Ciampoli, M., Millpress, Rotterdam.