COMPDYN 2013
4th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, V. Papadopoulos, V. Plevris (eds.)
Kos Island, Greece, 12–14 June 2013

EVALUATION OF THE EFFECT OF MODELING UNCERTAINTIES ON THE SEISMIC RESPONSE OF EXISTING MASONRY BUILDINGS

M. Rota[1], S. Bracchi[2], A. Penna[1,3] and G. Magenes[1,3]

[1] European Centre for Training and Research in Earthquake Engineering
via Ferrata, 1 I-27100 Pavia, Italy
maria.rota@eucentre.it

[2] ROSE Programme, UME School, IUSS Pavia
via Ferrata, 1 I-27100 Pavia, Italy
stefano.bracchi@umeschool.it

[3] University of Pavia, Department of Civil Engineering and Architecture
via Ferrata, 3 I-27100 Pavia, Italy
{andrea.penna, guido.magenes}@unipv.it

Keywords: modeling uncertainty, masonry buildings, seismic assessment, pushover analysis, equivalent-frame modeling.

Abstract. *An accurate seismic assessment of existing masonry buildings requires consideration of the sources of uncertainty and their effect on the seismic response. Among them, uncertainties related with modeling assumptions are definitely non-negligible. They include the choice of the modeling approach (continuum finite elements, equivalent frame, etc…), the analysis type (static/dynamic, linear/nonlinear) and the different options followed in defining the model. This work provides a quantitative evaluation, in probabilistic terms, of the effect of modeling uncertainties on the seismic response, in terms of the peak ground acceleration corresponding to the attainment of predefined limit states. Nonlinear static analysis was used, considered as the reference method to be adopted for the seismic assessment of existing buildings. The equivalent-frame macro-element approach was selected, being a satisfactory compromise between computational effort and accuracy in the results. Different possible choices regarding definition of the geometry of the equivalent frame, distribution of loads among the piers, distribution of loads on the floor systems, modeling of masonry spandrels, degree of coupling between orthogonal walls, definition of the cracked stiffness of structural elements, were considered. To quantify their effect on the response, a logic tree approach was followed, assigning a value of probability to each choice and hence obtaining the distribution of the acceleration corresponding to the selected limit states. This procedure was applied to a prototype building, for which a quantitative measure, in probabilistic terms, of the dispersion in the results due to the considered modeling uncertainties was evaluated.*

1 INTRODUCTION

Masonry structures constitute an important part of the existing building stock in many world countries, both for residential use and to host economical activities and very important social functions. The majority of these buildings was not conceived to survive earthquakes and it hence needs a seismic safety assessment, as demonstrated by the recent Italian seismic events in the Abruzzi region and in Emilia. It is hence particularly important that the seismic assessment methodology is accurate, leads to more reliable results as knowledge on the structure (and hence cost) increases and allows to correctly take into account all the uncertainties involved in the process.

The evaluation of the seismic safety of existing buildings is currently carried out with conventional methods, like those adopted in the most advanced codes, in which knowledge and uncertainties are accounted for in an all-in way. In particular, the approach included in Eurocodes [1] and in the Italian building code [2] simply accounts for knowledge by defining three discrete levels of knowledge and associating to each of them a value of confidence factor, to be applied as a reduction of material strengths.

Previous literature works have demonstrated that this methodology provides in many cases inconsistent results, both for the case of reinforced concrete [3] and masonry [4] buildings. Also, this approach does not take into account all the other sources of uncertainty, such as for example those related with the selected modeling strategy and options, the adoption of deterministic thresholds identifying the ultimate element drifts, the epistemic uncertainty in the definition of the seismic action.

A previous work [5] proposed a probabilistic methodology for quantifying the effects of these sources of uncertainty on the results of nonlinear static analyses, based on the definition of coefficients (called variability factors), which are calibrated based on a logic tree approach. However, in that work, only a preliminary estimate of the effects of different modeling assumptions was reported, taking into account only some of the possible hypotheses related with the equivalent-frame macro-element modeling of masonry buildings.

Still keeping the same methodological approach proposed in [5], this paper presents a more accurate evaluation of the effects of such modeling uncertainties, trying to consider all the possible analysis options, although only concentrating on nonlinear static analysis and on the equivalent-frame macro-element modeling strategy. The choice of considering only nonlinear static analysis derives from the consideration that this analysis method can be currently considered as the best-established approach for the seismic assessment of masonry buildings. Indeed, although nonlinear dynamic analyses are universally considered the most accurate analysis technique, their application in the engineering practice is still quite rare (e.g. [6]), also due to the high computational effort and to objective difficulties related for example with the definition of seismic input in terms of appropriate acceleration time histories (e.g. [7], [8]) and the identification of appropriate damage limit states (e.g. [9]).

The proposed methodology is applied to a case study building, which is described in the following section and corresponds to one of the buildings considered in [5] to allow comparison of the results obtained.

2 CASE STUDY BUILDING AND MODELING APPROACH

The proposed methodology was applied to a three-story building, with an approximately square plan configuration (Figure 1). This building, which is one of the structures (building H) considered in [5] for a preliminary estimate of the effect of modeling uncertainty, is realized with “roughly dressed stone masonry with good bonding”, according to the predefined masonry typologies reported in Table C8.A.2.1 of the Commentary to the Italian code [10]. Mechanical parameters were hence assumed equal to the central values of the intervals reported in [10], neglecting their spatial variability within structural elements. No confidence factor was applied, assuming a perfect knowledge on material properties (level of knowledge at least equal to 3). This building has rigid diaphragms and its construction details are such to guarantee a global type of response with no premature activation of local (out-of-plane) failure mechanisms. To reach this scope, simple construction details are sufficient, as demonstrated for example in [11] and [12].

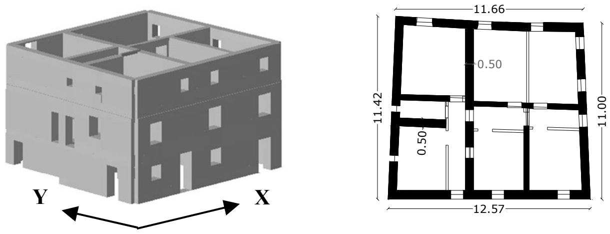


Figure 1: 3D model view and plan view of the considered case study building.

As mentioned in the introduction, the choice of the software used for carrying out all the analyses can strongly affect the results obtained. In this work, a single software has been selected, without hence taking into consideration the effect on the results of the adoption of different analysis programs and modeling approaches. The selected software, called Tremuri, is based on the macro-element approach and allows execution of nonlinear static and nonlinear dynamic analyses of entire masonry buildings, by means of an equivalent-frame idealization of the structure. Details about this program and its algorithm can be found elsewhere (e.g. [13], [14]).

3 CONSIDERED MODELING OPTIONS

As already discussed, in this work only nonlinear analyses were performed and a single equivalent frame macro-element model was adopted. Under these hypotheses, the main modeling uncertainties, i.e. the different options that the engineer has to select when modeling the building, can be identified as being related with:

- Definition of the geometry of the equivalent frame, i.e. identification of masonry piers, definition of rigid and deformable portions
- Definition of cracked versus initial stiffness
- Modeling of masonry spandrels
- Distribution of loads on the horizontal diaphragms
- Distribution of vertical loads among the different masonry piers
- Degree of coupling between orthogonal walls

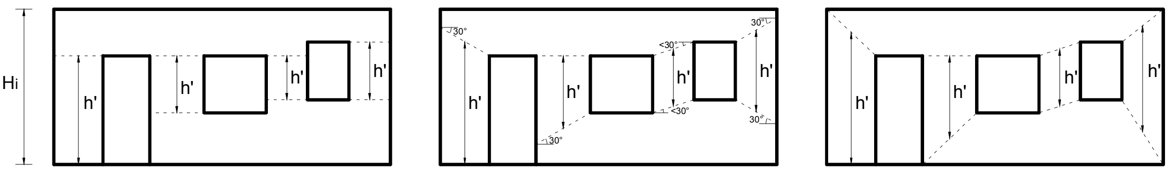
Another possible source of uncertainty is related with the capacity models used and, in particular, with the criterion used to evaluate the shear strength. Among the different criteria available in the literature, only the diagonal shear cracking failure criterion (proposed by Turnsek and Cacovic [15]) was adopted, as suggested in [10]. Additional uncertainty may be related with the definition of the effective length of tie beams, but this was not taken into consideration as the selected stone masonry building prototype (which is actually a real building) does not have any tie beam. The adopted software does not model the out-of-plane wall stiffness. Consideration of this additional stiffness could add an additional uncertainty to the results.

Each of the considered modeling options corresponds to one of the branches of the logic tree and hence its influence was evaluated separately.

In a previous paper [5], a preliminary estimate of the variability factor due to modeling uncertainties was carried out by considering only some of these sources of uncertainty, i.e. identification of masonry piers, distribution of loads on the floor systems, modeling of masonry spandrels and definition of cracked versus initial stiffness. This paper aims at providing a more precise quantification of the effects of modeling uncertainties, by considering all the elements listed above.

3.1 Definition of the height of masonry piers

Regarding the definition of the effective height of the piers, *h'*, three different criteria were considered, each one with the same probability of being selected. According to the first criterion (Figure 2 (a)), the pier height coincides with the minimum clear height between the adjacent openings. The second criterion (Figure 2 (b)) assumes a maximum 30° inclination of the cracks starting from the opening corners and consistently provides an increased height for the external piers. This was also the initial hypothesis used by Dolce [16] for the definition of the equivalent height of models based on the story mechanism. The third criterion (Figure 2 (c)), default criterion of the automatic mesh generator included in the 3muri program [14], provides an effective height equal to the average height of the adjacent openings (story height in case of external openings).



(a) (b) (c)

Figure 2: Effective pier height, *h'*, evaluated assuming three different discretization criteria: (a) minimum clear height, (b) effective height assuming a maximum inclination of cracks, and (c) average height of adjacent openings (after [5]).

3.2 Definition of cracked versus initial stiffness

A typical way of accounting for cracked material properties consists in reducing the values of the elastic moduli *E* and *G*, as also suggested in [10], which reports uncracked values of the elastic moduli *E* and *G* for different masonry typologies and suggests to appropriately reduce them to account for cracked conditions in the structure.

In this work, three values of the ratio of cracked versus initial (uncracked) stiffness were assumed, corresponding to 100%, 75% and 50%, with assigned probabilities equal to 45%, 10% and 45%, respectively. The first option may appear in contrast with the indications of [10], but it was considered reasonable for those cases in which the engineer believes the values reported in the table are already low if compared with the results of experimental tests. 50% is a reduction coefficient which is often adopted in the engineering practice and it also constitutes the default of several analysis programs . Finally, the value of 75% is the most consistent with experimental results (e.g. [17], [18], [19]) and therefore it is probably, in many cases, the most appropriate choice. This value was given a low probability, as it is believed that only a small percentage of engineers would resort to experimental results for the evaluation of this reduction coefficient.

3.3 Modeling of masonry spandrels

The degree of coupling exerted by masonry spandrels can strongly affect the buildings' structural response (e.g. [20], [21]). In this study, two extreme options were considered, which correspond to those most commonly adopted by structural analysis programs for the seismic assessment of masonry buildings. The two options were assumed to be equally probable.

In the first case, spandrels are modeled as deformable beams belonging to the equivalent frame and their lateral strength is determined either based on the compression forces deriving from the analysis (reliable only for walls not connected to rigid diaphragms) or on the strength of the tension resisting elements. The latter applies when tie rods or tie beams are present in parallel to the spandrels. In the second option, spandrels are modeled as axially rigid trusses simply coupling the horizontal displacements of the piers (cantilever model).

3.4 Distribution of loads on the floor systems

Two possible alternatives (with the same probability) were considered for the distribution of loads on floors and roofs, assuming these diaphragms are one-way floor systems. According to the first option, 100% of the load is acting on the walls crossing the direction of spanning of the floors, whilst in the second option this percentage is reduced to 75%, assuming that also walls parallel to the direction of spanning carry part of the loads acting on the floor (approximated as 25% of the total load).

3.5 Distribution of vertical loads among the different masonry piers

As well known, the lateral strength of masonry piers is strongly dependent on the applied vertical force which governs both shear and flexural strength criteria and hence the expected failure mode. Tributary floor areas can be defined in order to calculate the vertical load applied from the floor system to each pier. Different levels of refinement can be adopted in calculating the in-plane eccentricity of the vertical load acting on each masonry pier. The different assumptions can lead to different pushover analysis results in terms of both base shear and displacement capacity.

The following three criteria for calculating the longitudinal eccentricity of the vertical force transmitted by floors to walls have been considered, with an equal probability of being chosen:

a) the vertical load is concentrated at the center middle of each pier (left part of Figure 3) with no in-plane eccentricity;
b) the vertical load is applied in the point corresponding to the resulting vertical force transmitted by the floor elements pertaining to the tributary area (maximum longitudinal eccentricity);
c) the floor area is subdivided into strips (or equivalent joists) parallel to the spanning direction. Each strip is considered to distribute half of the vertical load corresponding to its area to the two intersecting walls (loading points corresponding to the strip ends). In case the strip directly loads a masonry pier, the load both contributes to the applied vertical force and bending moment based on the eccentricity of the loading point with respect to the pier center. The strip loads applied to beams or spandrels are instead reported to the beam/spandrel ends with an inverse proportionality with the distances between the loading point and the beam ends. The sum of all strip contributions at the beam-pier edge is then reported as a vertical load contribution to the pier center plus a bending moment obtained multiplying the load by half of the pier length. As a result of this more general approach, which was in-

cluded in the 3muri program also to account for any floor spanning direction [14], the load eccentricity is reduced with respect to criterion b).

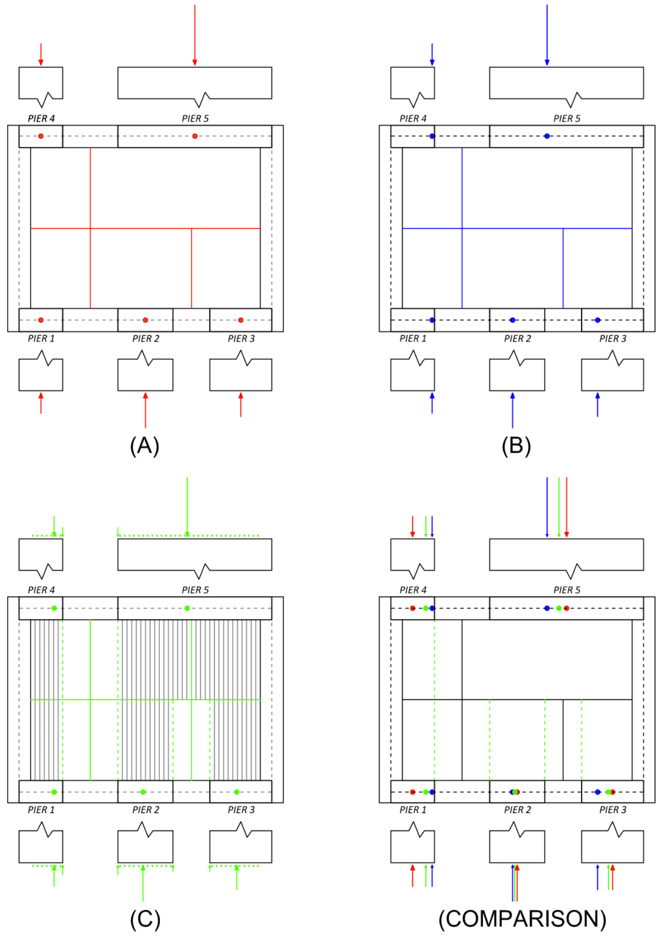


Figure 3: Three criteria considered for the evaluation of longitudinal eccentricity of vertical loads on masonry piers: (a) load centered on masonry piers, (b) eccentric load, and (c) strip model included in 3Muri.

3.6 Degree of coupling between orthogonal walls

For what concerns the degree of coupling between orthogonal walls, two criteria have been considered. According to the first one, the two orthogonal walls are connected by an infinitely stiff element providing a kinematic restraint. In this way the beam has a flange effect. This is the default's option of the Tremuri computer program and it was given a probability of 75% as it is considered to be the most correct modeling approach. Another alternative was also considered (with a probability of 25%), according to which the two orthogonal walls are connected by a link element, which means that there is no flexural and shear coupling between the walls.

4 ANALYSIS PROCEDURE

Two different force distributions (e.g. mass proportional and first mode distribution) were used in nonlinear static analyses, as indicated by several codes (e.g. [1] and [2]) to account for the dynamic response in the different phases of damage evolution. These force distributions were applied along two orthogonal directions (both positive and negative), considering the presence of the accidental eccentricities introduced by the codes (e.g. [22] and [2]) to account for uncertainties in the location of masses. Since the building has rigid floors, the choice of the control node is not very critical and one of the nodes belonging to the top level was selected.

Three limit states were considered, i.e. the two required by the Italian building code and indicated as life safety (ULS) and damage limitation (DLS) limit states, and the operational (OLS) limit state, which however is not explicitly required by the code for the building under study. The seismic input was defined by means of the elastic horizontal acceleration response spectrum of Type 1 proposed by EC8-1 [22] for rigid soil conditions (ground type A). The values of the ultimate shear and flexural drift at the element level were set equal to the values indicated in the Italian code, i.e. 0.4% and 0.6% respectively. As discussed in [5] these values are also affected by some uncertainty, which should be considered by defining an appropriate probability distribution. However, the purpose of this work is simply to evaluate modeling uncertainties and therefore all other sources of uncertainty are neglected.

The logic tree approach was used to evaluate the influence of modeling uncertainties, as discussed in the following section. Structural capacity was calculated according to the N2 method [23].

4.1 Definition of the logic tree and statistical treatment of the results

Similarly to what done in [4] and [5], the assessment of the structure was simulated by taking into account the effect on the assessment results of the possible choices related to modeling uncertainties. The different options were schematized in the form of a logic tree (reported in Figure 4), with each branch of the tree having a different probability of being chosen and each leaf corresponding to the results of the analysis carried out with the assumptions corresponding to the path followed within the tree. The definition of the probabilities associated with the different choices within the logic tree is subjective and it was based on the relative likelihood of each choice, determined based on engineering judgment, as discussed in the previous sections for each considered option.

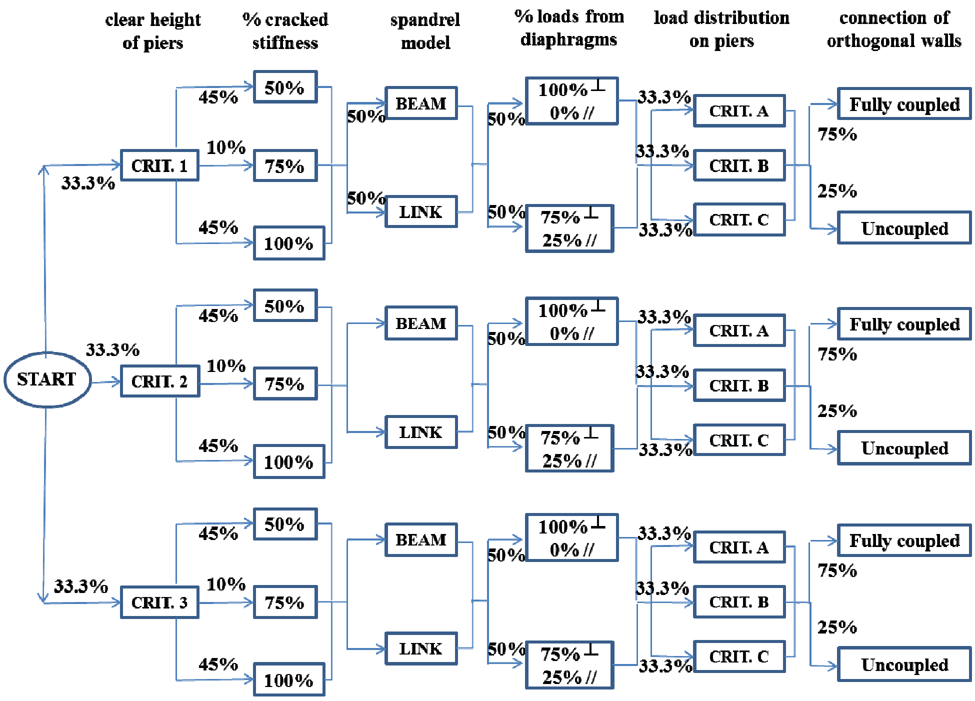


Figure 4. Logic tree followed for the definition of the effects of modeling uncertainties – the percentage values associated with the different braches indicate their probability of being selected.

Each analysis hence provided a value of the acceleration corresponding to the achievement of a given limit state and an associated probability obtained as the product of all the probabilities of the different branches followed. A histogram of values was then constructed and the corresponding cumulative distribution was fitted by a lognormal distribution, with parameters determined using the Levenberg e Marquardt ([24], [25]) nonlinear regression algorithm. As previously mentioned, three different limit states were considered and they were all defined according to the indications of [2] and [10]). The acceleration corresponding to the attainment of the life safety limit state (indicated as ultimate limit state, ULS) was obtained by applying the N2 method [23] reported in Annex F of [22] and in [2]. As specified in [10], for the case of nonlinear static analysis, the ULS is identified as the displacement corresponding to a 20% strength deterioration with respect to the maximum strength. To identify this point, the force-displacement curve obtained from the analysis is converted into the curve of an equivalent single degree of freedom system, which is then approximated by a bilinear curve. This last step was carried out according to the indications of [2]. With reference again to nonlinear static analysis, the damage limitation state (DLS) is identified as the minimum between the displacement corresponding to the maximum base shear and that for which the relative displacement of two adjacent stories exceeds $0.003h$, where h is the inter-story height [10]. The operational limit state (OLS) is instead identified as the displacement for which the relative displacement of two adjacent stories exceeds 2/3 of the value corresponding to DLS. An additional condition was also considered, imposing the displacement corresponding to OLS to be not larger than that corresponding to DLS. With these values of displacement it was then possible to derive the displacement of the equivalent single-degree-of-freedom system and then the acceleration corresponding to the attainment of these limit states.

From the probability distribution of the accelerations corresponding to the attainment of each damage state, the variability factor for that damage state was calculated as the ratio of the value corresponding to the 5th percentile to the mean value of acceleration, i.e.:

$$\alpha = \frac{a_{g,5\%}}{a_{g,mean}} \tag{1}$$

This variability factor is a measure of the dispersion of the acceleration values with respect to the mean value of the distribution and hence of the variability in the results caused by the considered modeling uncertainties.

5 RESULTS

This section summarizes the results obtained from the analyses of the considered building prototype and presents some comparisons with the results of the preliminary estimate of the effects of modeling uncertainties presented in [5].

Figure 5 shows the pushover curves obtained from analyses in the two perpendicular directions indicated in Figure 1 as X and Y. Each curve reported in the plot corresponds to the analysis providing the minimum value of the acceleration corresponding to the attainment of the ultimate limit state and derives from one of the branches of the logic tree (i.e. it corresponds to one set of modeling options), for a total of 216 curves. The dispersion of these pushover curves is significant in both directions.

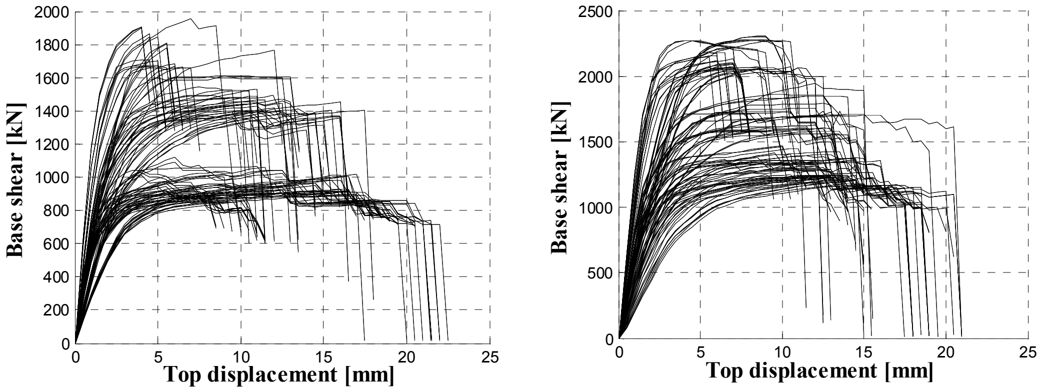


Figure 5. pushover curves corresponding to the minimum $a_{g,ULS}$ in direction X (left) and Y (right).

Figure 6 shows instead the cumulative distribution of the values of acceleration corresponding to the attainment of the ultimate limit state in the two directions and their lognormal approximations, whilst Figure 7 shows the lognormal approximations obtained for the three considered limit states.

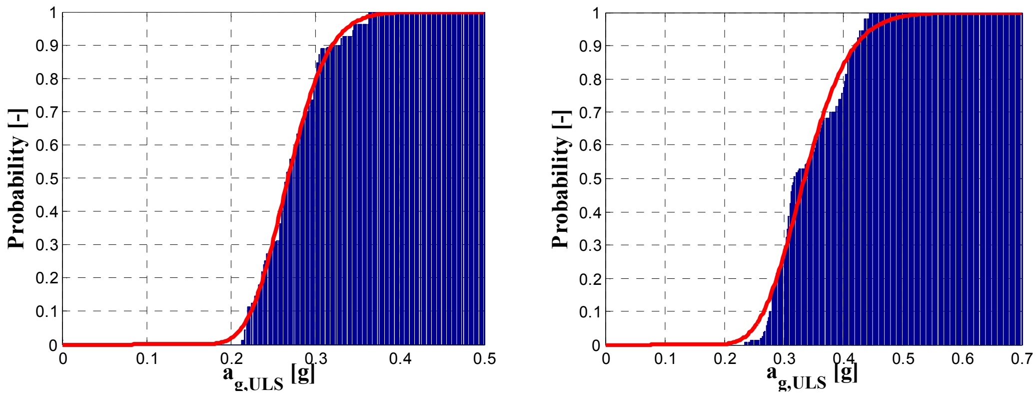


Figure 6. Cumulative distribution and lognormal approximation of the values of $a_{g,ULS}$ in direction X (left) and Y (right).

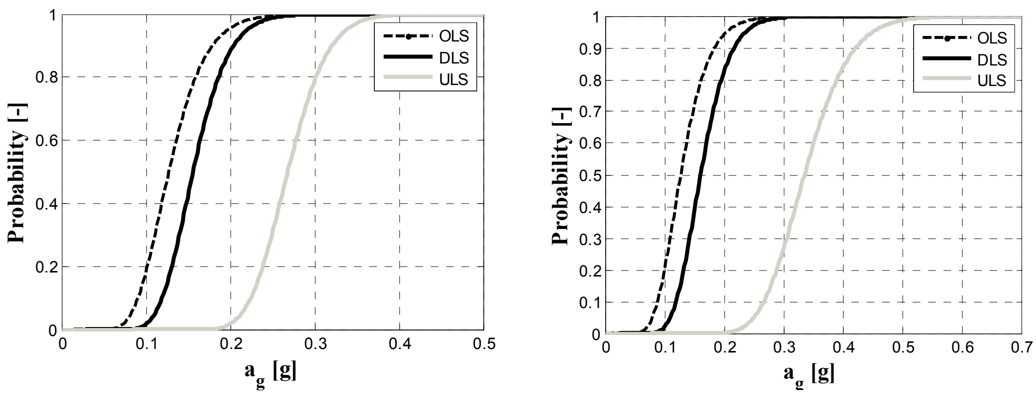


Figure 7. Lognormal approximation of the cumulative distribution of the acceleration capacity for the three considered limit states in the X (left) and Y (right) directions.

Comparison with the results of the preliminary study, whose assumptions are discussed in [5], shows that in the case presented in this paper the dispersion in pushover curves is more significant, as could be expected since additional sources of uncertainty have been included. Nevertheless, a comparison of the parameters characterizing the lognormal approximation curves reported in Figure 6 with those obtained in the preliminary study shows that they are not significantly different, as reported in Table 1. In particular, in both directions, the log-normal standard deviation (representing variability in the results) is higher than in the preliminary estimate for the OLS and DLS, whilst it is lower for the ULS. This may be due to the limitation on the maximum allowable spectrum reduction factor imposed by the Italian code ($q^{*}\leq 3$), which governs the ultimate displacement capacity of the structure, hence reducing the dispersion in the analysis results.

Limit state	This study				Preliminary evaluation			
	Direction X		Direction Y		Direction X		Direction Y	
	μ	σ	μ	Σ	μ	σ	μ	σ
OLS	-2.064	0.266	-2.064	0.288	-1.935	0.249	-1.905	0.263
DLS	-1.860	0.210	-1.830	0.234	-1.756	0.198	-1.702	0.219
ULS	-1.317	0.140	-1.093	0.179	-1.206	0.155	-1.000	0.193

Table 1: Comparison of the parameters of the normal distributions associated with the lognormal approximations obtained in this study and from the preliminary evaluation of modeling uncertainties reported in [5].

The variability factors were calculated based on the distribution of the acceleration corresponding to the attainment of the different limit states. For each limit state, two values of α_{mod} were calculated, one for each direction. Table 2 shows a comparison between the values of the variability factors obtained in this study and those obtained in the preliminary evaluation carried out with the assumptions discussed in [5]. It can be noted that, in both directions, the variability factor accounting for modeling uncertainties obtained in this study are lower than in the preliminary evaluation for the OLS and DLS, whilst the opposite occurs for the ultimate limit state. This indicates that, as expected, consideration of additional sources of modeling uncertainty tends to increase the dispersion in the results for the damage limit states, hence leading to lower values of the variability factors. The opposite occurs for the ultimate limit state, for the same reason discussed above regarding the lognormal parameters.

Limit state	This study		Preliminary evaluation	
	Direction X	Direction Y	Direction X	Direction Y
OLS	0.6237	0.5971	0.6435	0.6265
DLS	0.6929	0.6626	0.7078	0.6816
ULS	0.7871	0.7335	0.7653	0.7148

Table 2: Values of α_{mod} obtained for the three limit states considered in this study and in the preliminary evaluation whose hypotheses are discussed in [5].

6 CONCLUSIONS

This paper presented the results of a quantitative estimate of the effects of modeling uncertainties on the seismic response of a stone masonry building, evaluated by means of nonlinear static analyses with an equivalent frame macro-element approach. The effect of the different modeling choices on the seismic response, represented by the peak ground acceleration corresponding to the attainment of predefined limit states, is considered by means of a logic tree approach, in which each option corresponds to a branch of the tree, with a given probability of being selected.

From the probability distribution of the values of acceleration obtained from the analyses, values of the so-called variability factor were obtained. This factor, defined as the ratio between the value corresponding to the 5^{th} percentile of the distribution and the mean value, provides a quantitative measure of the dispersion in the analysis results due to consideration of the different modeling options.

The study concentrated on nonlinear static analysis, as it is considered the best-established method currently adopted in Italy for the seismic assessment of existing masonry buildings. Moreover, only the equivalent-frame macro-element modeling strategy has been adopted.

The results obtained for a case study building seem to indicate that the effect on the seismic response of masonry buildings of the uncertainty in modeling options is definitely non-negligible. Nevertheless, the methodology presented in this paper needs to be applied to other prototype buildings, such as for example those presented in [5], in order to derive more general indications on the quantitative estimate of the variability factor accounting for modeling uncertainty. This factor could then be adopted in the probabilistic approach proposed in [5], which allows carrying out the seismic assessment of masonry buildings considering the different sources of uncertainty involved in the problem and overcoming some of the main limitations of the current code approach adopted in Italy and in Europe.

ACKNOWLEDGEMENTS

This work was carried out within the framework of the 2012-14 EUCENTRE Executive Project e1.b “Seismic assessment of masonry buildings accounting for the knowledge level and the different sources of uncertainty” funded by the Italian Department of Civil Protection. The valuable help of Mr. Marco Tondelli is gratefully acknowledged.

REFERENCES

[1] EN 1998-3. Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings. CEN, Bruxelles, 2005.

[2] NTC08 Decreto Ministeriale 14 Gennaio 2008: “Norme tecniche per le costruzioni,” Ministero delle Infrastrutture. S.O. n.30 alla G.U. del 4.2.2008, No. 29, 2008.

[3] P. Franchin, P.E. Pinto, P. Rajeev, Confidence factor? *Journal of Earthquake Engineering*, **14**(7), 989-1007, 2010.

[4] M. Tondelli, M. Rota, A. Penna, G. Magenes, Evaluation of uncertainties in the seismic assessment of existing masonry buildings. *Journal of Earthquake Engineering*, **16**(S1), 36-64, 2012.

[5] M. Rota, A. Penna, G. Magenes, A framework for the seismic assessment of existing masonry buildings accounting for different sources of uncertainty. *Submitted to Earthquake Engineering and Structural Dynamics*, 2013.

[6] A. Penna, M. Rota, A. Mouyiannou, G. Magenes, Issues on the use of time-history analysis for the design and assessment of masonry structures. *4th International Confer-*

ence on Computational Methods in Structural Dynamics and Earthquake Engineering (COMPDYN), Kos Island, Greece, 2013.

[7] M. Corigliano, C.G. Lai, M. Rota, C.L. Strobbia, ASCONA: Automated Selection of COmpatible Natural Accelerograms. *Earthquake Spectra*, **28**(3), 965-987, 2012.

[8] M. Rota, E. Zuccolo, L. Taverna, M. Corigliano, C.G. Lai, A. Penna, Mesozonation of the Italian territory for the definition of real spectrum-compatible accelerograms. *Bulletin of Earthquake Engineering*, **10**(5), 1357-1375, 2012.

[9] A. Mouyiannou, M. Rota, A. Penna, G. Magenes, Identification of suitable limit states from nonlinear dynamic analyses of masonry structures. *Submitted to Journal of Earthquake Engineering*, 2013.

[10] MIT Ministry of Infrastructures and Transportation Circ. C.S.Ll.Pp. No. 617 of 2/2/2009 "Istruzioni per l'applicazione delle nuove norme tecniche per le costruzioni di cui al Decreto Ministeriale 14 Gennaio 2008," Consiglio superiore dei lavori pubblici. S.O. n.27 alla G.U. del 26.02.2009, No. 47, 2009.

[11] G. Magenes, A. Penna, I. Senaldi, M. Rota, A. Galasco, Shaking table test of a strengthened full scale stone masonry building with flexible diaphragms. *Submitted to International Journal of Architectural Heritage*, 2013.

[12] I. Senaldi, G. Magenes, A. Penna, A. Galasco, M. Rota, The effect of stiffened floor and roof diaphragms on the experimental seismic response of a full scale unreinforced stone masonry building. *Submitted to Journal of Earthquake Engineering*, 2013.

[13] A. Penna, S. Lagomarsino, A. Galasco, A nonlinear macro-element model for the seismic analysis of masonry buildings. S*ubmitted to Earthquake Engineering and Structural Dynamics*, 2013.

[14] S. Lagomarsino, A. Penna, A. Galasco, S. Cattari, TREMURI program: an equivalent frame model for the nonlinear seismic analysis of masonry buildings. *Submitted to Engineering Structures*, 2013.

[15] V. Turnsek, F. Cacovic, Some experimental results on the strength of brick masonry walls. *2nd International Brick Masonry Conference*, Stoke on Trent, 1971.

[16] M. Dolce, Schematizzazione e modellazione per azioni nel piano delle pareti, *L'Industria delle Costruzioni*, 44-57, 1991 (in Italian).

[17] G. Magenes, P. Morandi, A. Penna, In-plane cyclic tests of calcium silicate masonry walls. *14th International Brick Block Masonry Conference*, Sydney, Australia, 2008.

[18] A.A. Costa, A. Penna, G. Magenes, Seismic performance of autoclaved aerated concrete (AAC) masonry: from experimental testing of the in-plane capacity of walls to building response simulation. *Journal of Earthquake Engineering* **15**(1), 1-31, 2011.

[19] M. Mandirola, A. Penna, M. Rota, G. Magenes, Experimental assessment of the shear response of autoclaved aerated concrete (AAC) masonry with flat truss bed-joint reinforcement. *15th International Brick Block Masonry Conference*, Florianopolis, Brazil, 2012.

[20] M. Tomazevic, Earthquake-resistant design of masonry buildings, *Imperial College Press, Series on innovation in structures and constructions*, 1999.

[21] G. Magenes, A. Della Fontana, Simplified non-linear seismic analysis of masonry buildings. *Proc. British Masonry Society* **8**, 190-195, 1998.

[22] EN 1998-1 Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings. CEN, Bruxelles, 2004.

[23] P. Fajfar, A nonlinear analysis method for performance-based seismic design. *Earthquake Spectra*, **16**(3), 573–592, 2000.

[24] K. Levenberg, A method for the solution of certain non-linear problems in least squares. *Quarterly of Applied Mathematics*, **2**(2), 164–168, 1944.

[25] D.W. Marquardt, An algorithm for the least-squares estimation of nonlinear parameters. *SIAM Journal of Applied Mathematics*, **11**(2), 431–441, 1963.