SAHC2014 – 9th International Conference on
Structural Analysis of Historical Constructions
F. Peña & M. Chávez (eds.)
Mexico City, Mexico, 14–17 October 2014

SEISMIC ASSESSMENT OF MASONRY BUILDINGS CONSIDERING DIFFERENT UNCERTAINTIES

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Keywords: seismic assessment; masonry buildings; uncertainty; confidence factors; modelling uncertainties; knowledge levels.

Abstract. *This works moves from considerations about the inadequacy of the approach currently adopted by the Eurocode 8 Part 3 and the Italian building code for the seismic assessment of existing buildings. This approach is based on the identification of a discrete number of knowledge levels, for which deterministic values of the confidence factors are defined. Such factors need to be applied to material strength only, and they should implicitly account for all the knowledge-based uncertainties involved in the seismic assessment. Previous literature works have shown that this formulation often produces inconsistent results, both for reinforced concrete and for masonry buildings. Also, several additional sources of uncertainty that have an effect on the results of the assessment are not correctly taken into account. This work proposes a probabilistic methodology for the assessment of existing masonry buildings by means of nonlinear static analysis, taking into account the different sources of uncertainty involved in a simple yet rigorous way. In particular, this approach considers the uncertainties related with modelling assumptions, with the identification of deterministic thresholds for the definition of the ultimate element drift ratios and with the acquired knowledge on material properties. This simple approach, also including an alternative formulation of the confidence factors related with material properties, allows to obtain results which are consistent with the acquired level of knowledge and correctly accounts for the different sources of uncertainty without requiring to carry out any stochastic nonlinear analysis. Confidence factors accounting for the uncertainty in the material properties, to be applied directly to building capacity rather than only to material strength, are calibrated based on a Bayesian updating approach, taking into account the experimental results of in-situ tests.*

1 INTRODUCTION

In the current approach of the Italian code [1] and Eurocode 8 Part 3 [2] for the seismic assessment of existing masonry buildings, discrete knowledge levels (KLs) are defined based on the level of knowledge on geometry, construction details and material properties. With each KL, a different value of the confidence factor (CF) is associated, which must be applied as a reduction of material strengths. These factors are assumed to account for all the uncertainties involved in the seismic assessment of masonry buildings, by reducing their effect on the results as the level of knowledge increases.

This work moves from the results of previous studies, which have shown that the current code approach does not always produce consistent results, at least in the case of nonlinear static analysis, both for reinforced concrete ([3], [4], [5]) and masonry buildings [6]. In particular, for the case of masonry buildings, the code approach showed several limitations, starting from the consideration that in some cases it is not possible to reach the highest level of knowledge (KL3), as it is practically impossible or too expensive to perform the destructive in-situ tests required by the code for KL3. Even when it is possible to perform a single or few tests, as required by the code, the results are often not representative of the global mechanical characteristics of the structure and, definitely, they cannot be considered a significant statistical sample. A further limitation of the current code approach is related to the difficulty in the identification of the correct masonry typology among those considered in the commentary to the Italian code [7], which includes a list of the most common typologies and provides, for each of them, intervals of variation of the mechanical properties (compressive and shear strength, Young and shear modulus) or single values (specific weight). The identification of the correct typology is often difficult, particularly in the case of stone masonry and an incorrect identification of the typology could determine a systematic bias in the results, with respect to those of the ideal perfectly known structure.

Moreover, the application of the CFs as a reduction of material strengths produces in several cases un-conservative results [6]. This is particularly true when the selection of a different level of knowledge leads to the evaluation of a different collapse mechanism. Also, the physical meaning of the CFs is not clear, as an explicit link between these CFs and the analysis results is lacking (at least in case of nonlinear analysis, which is currently in Italy the reference method for the seismic analysis of masonry buildings).

This paper briefly describes a methodology for the seismic assessment of masonry buildings, taking into consideration all the sources of uncertainty in a simple yet rigorous way, previously proposed by the same authors [8]. This approach accounts for the uncertainty related with the knowledge and definition of the mechanical properties, the uncertainty related to the assumptions at the base of the numerical model of the building (modelling uncertainties), the uncertainty in the definition of numerical values for the displacement or deformation thresholds which identify the attainment of relevant limit states for masonry elements and the (epistemic) uncertainty related with the definition of the seismic input. The methodology also requires a new definition of the confidence factors accounting for material properties, whose definition and calibration constitute the main contribution of this paper. The CFs are applied to the values of structural capacity (acceleration corresponding to the attainment of the ultimate limit state, $a_{g,ULS}$) obtained from an analysis without any CF. The values of these new CFs for material properties are calibrated based on a Bayesian updating approach, taking into account the information on material parameters obtained from the experimental tests carried out at the higher KLs.

The study will refer to global analysis of the structures as, according to the Italian code [1], in the case of the analysis of local failure mechanisms the CF is applied directly to the capacity, which is therefore inversely proportional to the adopted value of CF. More in detail, reference will be made only to nonlinear static analysis, which is considered to be the best-established method for the seismic assessment of masonry buildings. Indeed, nonlinear dynamic analysis, despite being the most accurate analysis technique, often adopted for research purposes for masonry buildings (e.g. [9]), is still seldom used in the engineering practice, also due to objective difficulties related for example to the definition of seismic input in terms of appropriate acceleration time histories (e.g. [[10]-[12]]) and the identification of appropriate damage limit states (e.g. [13]).

A logic tree approach is used for a preliminary quantification of the effect of several sources of uncertainty on the seismic response of existing masonry buildings.

2 PROPOSED METHODOLOGY FOR THE SEISMIC ASSESSMENT OF MASONRY BUILDINGS

The proposed approach for the seismic assessment of masonry buildings is based on the determination of the peak ground acceleration corresponding to the attainment of a predefined limit state ($a_{g,LS}$) from a deterministic analysis, in which structural capacity is set equal to the demand imposed on the structure, to find the so-called performance point. The obtained value of acceleration for the selected limit state is then modified by means of “variability factors”, defined on purpose to account for the different uncertainties, and by CFs allowing to consider the KL on mechanical properties.

Assuming all uncertainties to be independent, the acceleration corresponding to the limit state LS can be obtained as:

$$a_{g,LS} = \alpha_{\text{mod}} \cdot \alpha_{LS} \cdot \frac{\bar{a}_{g,LS}}{CF_{mat}} = \alpha_{tot} \cdot \bar{a}_{g,LS} \tag{1}$$

where:

- α_{mod} and α_{LS} are the so-called “variability factors” taking into account the uncertainty related with modelling assumptions and with the identification of deterministic thresholds for the definition of the ultimate element drift ratios, respectively, and do not depend on the acquired level of knowledge
- $\bar{a}_{g,LS}$ is the acceleration corresponding to the attainment of a given limit state, obtained from the analysis without the application of any CF
- CF_{mat} is an appropriately defined confidence factor accounting for the acquired knowledge on material properties
- α_{tot} is, for each knowledge level, a unique safety coefficient obtained as the product of α_{mod}, α_{LS} and $\alpha_{mat} = 1/CF_{mat}$.

The calibration of the different terms of Equation (1) will be discussed in the following sections. Given a distribution of values for the acceleration corresponding to the attainment of a limit state, which is obtained by applying a logic tree approach to account for the different choices that an engineer would face in the assessment (as described in a following section), the variability factors are defined as the ratio of the value corresponding to the 5th percentile to the mean value of this distribution of values of acceleration, i.e.:

$$\alpha = a_{g,5\%} / a_{g,mean} \tag{2}$$

Therefore, these variability factors are 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 consideration of one of the sources of uncertainty listed above. The 5th percentile was selected, analogously to the definition of the characteristic values for actions and strengths typically adopted in structural design.

It is remarked that, with respect to the current code definition of the CFs, which are assumed to account for uncertainty related with knowledge on geometry, structural details and material properties, in the proposed approach the CF is only related to knowledge on material properties, as the first two sources of uncertainty are somehow already incorporated in the variability factor accounting for modelling uncertainty.

In [8] an additional source of uncertainty was considered, consisting in the epistemic uncertainty in the definition of seismic input. Although a preliminary evaluation of the effect of this uncertainty showed that its influence on the results of the assessment seems to be definitely non-negligible, this uncertainty has not been explicitly accounted for in the proposed assessment methodology. The main reason for this choice is that the decision on whether epistemic uncertainty in the definition of seismic input should be considered or not is not only related with the assessment of existing buildings but it should possibly also involve the design of new structures.

The methodology was applied to 8 building prototypes, which will be briefly presented in the next section. The different variability factors and the CFs on material properties were calibrated for these buildings, with reference to the case of nonlinear static analysis and equivalent-frame macro-element modelling, as discussed in the following.

3 EVALUATION OF THE EFFECT OF MODELLING UNCERTAINTY AND UNCERTAINTY IN THE DEFINITION OF LIMIT STATE THRESHOLDS

3.1 Considered prototype buildings

The different factors of the proposed methodology were evaluated with reference to 8 building prototypes, selected as representative samples of common configurations of existing stone masonry structures (in Italy). A view of the 3D model of each of these buildings is shown in Figure 1.

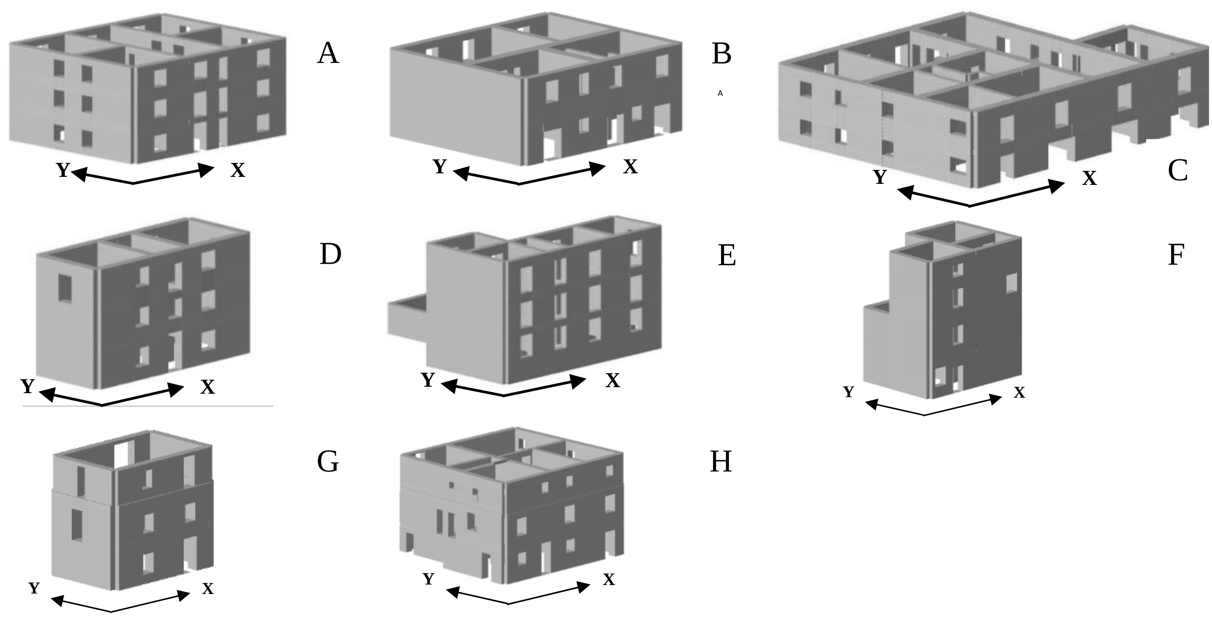

Figure 1: 3D models of the considered buildings, with indication of the two directions of analysis.

It was assumed that the geometry and the constructions details of the analyzed buildings were such to prevent local (out-of-plane) collapse mechanisms, and hence to guarantee a global type of response. Therefore only the global response of the buildings was considered, neglecting the analysis of the local mechanisms of collapse. All analyses were carried out using the research program TREMURI, whose algorithm is described in detail in [14] and [15].

All the buildings were assumed to be made of “roughly dressed stone masonry with good bonding”, a masonry typology reported in the commentary to the Italian code, and the mechanical parameters were assumed equal to the central values of the corresponding intervals reported therein. The buildings were assumed to be constituted by a single homogeneous material, i.e. the spatial variability of mechanical properties among structural elements was neglected.

All buildings were assumed to have rigid diaphragms in their plane, in order to exclude for the time being the issues related with nonlinear static analyses in case of flexible diaphragms (e.g. [6], [16]), which would potentially hide the effect of the other sources of uncertainty of interest for this study. With the assumption of rigid diaphragms, a node of the upper story was selected as the control node.

3.2 Adopted methodology of study

The different sources of uncertainty were assumed to be independent and therefore, when quantifying the effect of one of them, all the parameters related to the other sources of uncertainty were assumed to be deterministic. In particular, the seismic input was described by means of the EC8-1 [17] type 1 elastic acceleration response spectrum for soil A.

When estimating the effect of modelling uncertainties, deterministic values of the deformation limits corresponding to ultimate conditions for the shear and flexural failure modes were adopted. In particular, the values indicated in the commentary to the Italian code for in-plane element drift limits, i.e. 0.4% and 0.6% respectively for shear and flexural failures, were used. Similarly, when estimating the effect of uncertainty in the definition of these deformation limits, deterministic modelling assumptions were considered, as described in a following section.

As previously mentioned, this work concentrates on the case of nonlinear static analysis, carried out with two different force distributions, as indicated in the Italian code [1]. In particular, a mass proportional distribution and a first mode distribution were used. Forces were applied along the two perpendicular directions (positive and negative) indicated in Figure 1 as X and Y, considering the presence of the accidental eccentricity (positive, negative or null). Therefore, for each branch of the logic tree, 24 analyses were carried out and the capacity in each direction was identified as the minimum of the results of the 12 analyses performed in the considered direction.

The set of analyses on the structure resulting from each branch of the logic tree discussed in the following sections provided a value of the acceleration corresponding to a given limit state and an associated probability. A histogram of values was then constructed and the corresponding cumulative distribution was fitted by a lognormal distribution, with parameters determined using the Levenberg-Marquardt [18][19] nonlinear regression algorithm.

The acceleration corresponding to the attainment of the ultimate limit state (ULS) was evaluated according to the N2 method [20], also adopted in EC8-1 [17] and in NTC08 [1]. In case of nonlinear static analysis of masonry buildings, both [2] and [7] specify that the ultimate displacement capacity is evaluated at the point of the force-displacement curve corresponding to a strength degradation (after the peak) of 20% of the maximum value. The force-displacement curve obtained from the analysis is then converted into the curve of an equivalent single-degree-of-freedom (sdof) system, which is then approximated by a bilinear curve (examples of application of the bilinear approximation procedure can be found in [6] and [21]). This last step was carried out according to the indications of NTC08 [1]. The acceleration capacity corresponding to the ULS was then evaluated as discussed in detail in [8].

Two additional limit states were considered, i.e. damage limitation state (DLS) and operational limit state (OLS). According to [1], in case of nonlinear static analysis, the displacement capacity corresponding to these limit states has to be evaluated on the force-displacement curve at the following points:

- DLS: minimum displacement between the one corresponding to the maximum base shear and the one for which the relative displacement of two adjacent storeys exceeds 0.003 *h*, with *h* the inter-storey height;
- OLS: displacement for which the relative displacement of two adjacent storeys exceeds 0.002 *h* (i.e. 2/3 of the deformation limit value corresponding to DLS).

An additional condition was also considered, imposing the displacement corresponding to OLS not to be larger than that corresponding to DLS. With these values of displacement it was then possible to derive the displacement of the equivalent sdof system and then the acceleration corresponding to the attainment of these limit states.

The proposed values of the variability factors are obtained as the average of the results of the eight considered prototype buildings.

3.3 Variability factor accounting for modelling uncertainty

The assessment of the prototype structures was simulated using an approach similar to that followed in [6], taking into account the effect on the assessment results of the possible choices related to modelling uncertainties. The different options were schematized in the form of a logic tree 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 subjective probabilities associated with the different choices within the logic tree (weights of the different branches) were based on engineering judgment. In this preliminary work, the different choices at the same level of the tree were all assumed to have the same probability.

A preliminary estimate of the variability factor due to modelling uncertainties was derived considering the uncertainty related with identification of the effective height of masonry piers, distribution of loads on the floor systems, modelling of masonry spandrels and definition of cracked versus initial stiffness. The different modelling options considered are discussed in [8].

Table 1 summarizes the values of α_{mod} calculated for all the considered buildings and for the three limit states, together with their statistics. For each building and for each limit state, the value of α_{mod} reported in the table was evaluated separately for the two directions. This choice derives from assuming that analyzing the same building in two perpendicular directions actually is equivalent to analyzing two different structures. In practical terms, this corresponds to having a doubled sample of buildings. The values of α_{mod} vary between 0.61 and 0.96 for the ULS, between 0.50 and 0.85 for the DLS and between 0.44 and 0.85 for OLS. The results show that, in general, the dispersion in the results decreases (corresponding to higher variability factors) moving to more severe limit states, with the exception of building D and building G. This indicates that modelling uncertainties have a more pronounced effect on serviceability limit states and, in particular, on the OLS. The standard deviation and the coefficient of variation of the results decrease moving to higher limit states, indicating that the variability in the results from building to building is quite small for the ULS and higher for the serviceability limit states. The dispersion of the results obtained for the ULS is also reduced due to the limitation of the maximum behavior factor q^* which, according to the Italian code, cannot exceed the value of 3. In spite of this, the effect of modelling uncertainties on the ultimate limit state is definitely non-negligible.

Table 1: Values of α_{mod} obtained for the three limit states, for all buildings, and corresponding mean values, standard deviations and coefficients of variation.

Building		A	B	C	D	E	F	G	H	Mean	St. dev.	C.o.V.
α_{ULS}	X	0.964	0.877	0.885	0.707	0.861	0.854	0.907	0.765	0.794	0.098	12.4%
	Y	0.826	0.642	0.611	0.818	0.726	0.790	0.752	0.715			
α_{DLS}	X	0.589	0.677	0.753	0.846	0.512	0.734	0.837	0.708	0.693	0.113	16.3%
	Y	0.696	0.836	0.657	0.813	0.497	0.539	0.717	0.682			
α_{OLS}	X	0.498	0.589	0.753	0.846	0.458	0.734	0.837	0.644	0.655	0.143	21.8%
	Y	0.576	0.836	0.607	0.819	0.445	0.471	0.736	0.627			

3.4 Variability factor accounting for the uncertainty in the definition of limit states' thresholds

The current code approach provides deterministic values of the ultimate element drift in case of shear failure (0.4%) and flexural failure (0.6%). Nevertheless, experimental studies showed that the values of ultimate displacement for the different types of failure vary significantly depending on the masonry typology (e.g. [21], [22], [23]) and often present a significant dispersion even within a single masonry type (e.g. [24]). This suggested the need of investigating the effect on the results of the uncertainty in the definition of the limit state thresholds.

Differently from modelling uncertainties, the uncertainty in the definition of the ultimate drift thresholds obviously affects only the acceleration corresponding to the attainment of the ultimate limit state, whereas it does not have a significant effect on the serviceability limit states.

Values of the variability factors related with the definition of limit state thresholds were calculated following the procedure discussed in [8]. A non-perfect correlation between the shear drift δ_S and the flexural drift δ_F was assumed, considering an error random variable ε, i.e.:

$$\delta_F = (2+\varepsilon)\cdot\delta_S \tag{3}$$

with the shear drift δ_S considered as a random variable uniformly distributed between 0.3% and 0.5% and the error variable ε uniformly distributed within the interval

$$\frac{-0.2\%}{\delta_S} \le \varepsilon \le \frac{0.2\%}{\delta_S} \tag{4}$$

A value of δ_S and ε was randomly generated for each structural element of the model, hence allowing to take into account the element-to-element variability of these drift limits, even within the same building realized with the same masonry typology. The flexural drift δ_F was then obtained from Equation (3), with the additional constraint of $\delta_F \ge 0.5\%$.

Table 2 summarizes the values of α_{LS} obtained for the ultimate limit state and for all the considered buildings, together with their statistics. For each building, α_{LS} was evaluated separately for the two directions of analysis, for the same reasons discussed in the previous section.

The values of α_{LS} vary between 0.82 and 0.98 for the ULS, with a mean value of 0.925. The dispersion in the results is rather small, as confirmed by the coefficient of variation, which is smaller than 5%. Based on these results, it can be concluded that the effect on the seismic assessment of the uncertainty in the definition of limit state thresholds is smaller than that due to modelling uncertainty.

Table 2: Values of α_{LS} obtained for the ultimate limit state, for all buildings, and corresponding mean values, standard deviations and coefficients of variation.

Building		A	B	C	D	E	F	G	H	Mean	St. dev.	C.o.V.
α_{LS}	*X*	0.975	0.965	0.977	0.891	0.972	0.936	0.959	0.946	0.925	0.041	4.5%
	Y	0.924	0.893	0.902	0.889	0.823	0.900	0.917	0.931			

4 PROPOSED NEW DEFINITION AND CALIBRATION OF CONFIDENCE FACTORS ON MATERIAL PROPERTIES

Different approaches can be considered for an efficient definition of the CFs accounting for uncertainty on material properties, which would produce consistent results for the seismic assessment of masonry buildings by means of nonlinear static analyses, as a function of the acquired level of knowledge. Based on the preliminary considerations reported in [8], it was decided to apply this CF_{mat} to the value of acceleration corresponding to the attainment of a given limit state.

The value of CF_{mat} needs to take into account the level of knowledge reached on the material properties of the structure to be assessed and, in particular, the additional information obtained by in-situ tests for the higher levels of knowledge. To this aim, a Bayesian updating framework was developed for KL2 and KL3, allowing to modify the a-priori knowledge of the different mechanical parameters, taking into account any additional information acquired either experimentally or by exploiting empirical correlations of material parameters with other parameters that have been directly measured. Four material properties were taken into account, i.e. the elastic modulus E, the compressive strength f_m, the shear modulus G and the shear strength τ_0 and Bayesian inference was applied to the mean values of these properties.

4.1 Proposed methodology for Bayesian updating of material properties based on experimental results

At KL2, a single double flat-jack test is performed, obtaining a direct experimental measure of the elastic modulus E. At KL3, in addition to the test performed at KL2, diagonal compression tests are carried out (one to three tests), providing a direct measure of the shear strength τ_0. Hence the problem becomes to update the prior knowledge on the mean of the four considered mechanical properties based on the experimental measure of μ_E and, possibly, $\mu_{\tau 0}$. Since only two of these parameters can be directly measured, experimental correlations between the different mechanical properties were defined based on literature results and were used to update the a-priori knowledge on them. In particular, the ratio between E and f_m (called α) and the ratio between G and E (called β) were described by normal distributions with parameters defined based on literature results. In the absence of a reliable empirical correlation between τ_0 and any of the other three properties, a linear correlation between τ_0 and E (called η) was defined based on the intervals of values reported in [7].

After an experimental measure of μ_E (and also of $\mu_{\tau 0}$ at KL3) is obtained from the tests, the prior information on the different mechanical properties is updated to obtain the posterior distributions of the four mechanical properties. These are obtained from the joint distribution of the different parameters, by using standard mathematical expressions. The posterior distributions are a function of the prior information, the measures of the parameters obtained from the experimental tests and the dispersion associated with these measures due to error intrinsic of any type of test.

The values of mechanical properties to be used as input for the numerical model with the software TREMURI were taken equal to the expected (mean) values of the different properties obtained after Bayesian updating.

4.2 Calibration of preliminary values of the confidence factor on material properties

Simulated assessments were carried out by means of nonlinear static analyses with the software TREMURI, using the values of the mechanical properties defined as previously discussed using Bayesian updating. For the case of KL1, for which no experimental test is carried out and the masonry typology is simply identified based on visual inspection, the central values of the intervals reported in [7] for each mechanical property were used.

The values of the CF_{mat}, to be adopted within the proposed probabilistic, were then calibrated from the results of these simulated assessments, expressed in terms of the acceleration leading to the attainment of the ultimate limit state, $a_{g,ULS}$, and knowing the $a_{g,ULS}$ of the perfectly known structure, assumed to belong to the second typology of [7] (undressed stone masonry with facing walls of limited thickness and infill core). In particular, the values of the confidence factors on material properties were calibrated imposing that a selected percentile of the distribution of the values of $a_{g,ULS}$ obtained from the simulated assessment for each knowledge level (95% percentile for KL1, 90% for KL2 and 84% for KL3) was lower than a selected percentage of the value of $a_{g,ULS}$ of the perfectly known structure (70% for KL1, 84% for KL2 and 95% for KL3).

Figure 2 shows the values of CF_{mat} obtained for the first 7 buildings reported in Figure 1, analyzed separately in the two directions. Statistics of the values obtained are reported in Table 3.

The results obtained show that, as expected, the confidence factors tend to decrease as the knowledge level increases. Moreover, the statistics of the obtained values of CF_{mat} show that the variability of the values calculated for the different cases of study, represented by the standard deviation, decreases as the knowledge on the structure increases.

The application of the calculated confidence factors to the distributions of the capacity ratio (ratio between $a_{g,ULS}$ obtained from the simulated assessment and $a_{g,ULS}$ of the perfectly known structure) has the effect of reducing the amount of virtual analysts overestimating the $a_{g,ULS}$ of the reference structure. This approach guarantees that only a small and fixed percentage of analysts overestimates the $a_{g,ULS}$ of the real structure. Moreover, the proposed approach is consistent with the philosophy behind the code which is based on rewarding the attainment of a higher knowledge of the building with the application of a lower confidence factor, i.e. with a smaller degree of conservatism on the results of the analysis.

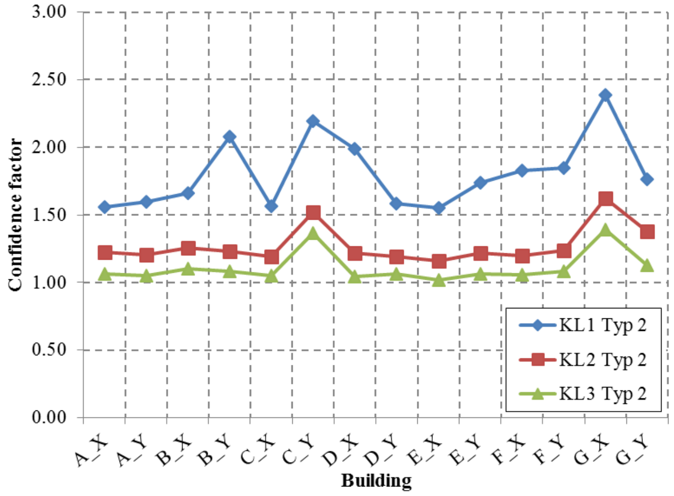

Figure 2: Values of CF_{mat} obtained for the different buildings, for the three KLs, in the case of perfectly known structure belonging to typology 2 of [7].

Table 3: Mean values, standard deviations and coefficients of variation of CF_{mat} obtained for the three KLs.

	KL1	**KL2**	**KL3**
Mean	1.81	1.28	1.11
St. dev.	0.26	0.14	0.12
C. o V.	0.15	0.11	0.10

5 COMMENTS AND CONCLUSIONS

This study moved from the results of previous works [6] highlighting several deficiencies of the Italian and European code approach for the seismic assessment of existing masonry buildings by means of nonlinear static analyses. In particular, the current code definition of the CFs, to be applied as a reduction of material strengths, proved to be often leading to inconsistent and un-conservative results. Moreover, the current code approach does not account for all the sources of uncertainty, assuming the CFs are able to account for all of them.

A previous paper by the authors [8], proposed an alternative approach allowing to solve some of the issues of the code methodology, without however radically changing the overall formulation, which was still based on the identification of KLs with associated values of a confidence coefficient. This alternative methodology allows consideration of all the sources of uncertainty (related to modelling options and to the application of deterministic values of the ultimate drift thresholds at the element level) by the application of so-called variability factors, which are calibrated based on a logic tree approach and aim to represent the dispersion in the results of the assessment due to each of these uncertainties. These variability factors can be applied to the final result of a deterministic analysis, without hence requiring the practitioner to carry out any stochastic analysis, but only to apply predefined coefficients to the results of the analysis.

The method also requires a new definition of the CFs on the mechanical properties of the structural materials, whose calibration is the main contribution of this paper. The use of these CF_{mat}, which need to be applied directly to the structural capacity expressed in terms of the acceleration corresponding to the attainment of the ultimate limit state, seems to provide results which are consistent with the philosophy on which the code is based, which aims at rewarding a higher level of knowledge by the need of applying a lower value of CF_{mat}. A procedure for the calibration of the values of CF_{mat}, based on the results of nonlinear static analyses carried out with material properties obtained after Bayesian updating of the mechanical parameters based on the results of experimental tests, is presented in this paper. The numerical values of CF_{mat} for the three knowledge levels proposed by the code were calibrated by imposing that a certain percentile of the virtual analysts underestimates a certain percentage of the $a_{g,ULS}$ of the perfectly known structure. The results obtained for the case in which the perfectly known structure belongs to typology 2 [8] are presented and seem to indicate that the proposed approach is able to produce reasonable results.

ACKNOWLEDGMENTS

This work was mainly carried out within different EUCENTRE Executive Projects in the years 2009-2014, funded by the Italian Department of Civil Protection. The authors would also like to acknowledge the contribution of Marco Tondelli and Giulia Grecchi, who helped carrying out some of the analyses discussed in the paper.

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