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SIMPLIFIED PROCEDURE FOR THE SEISMIC RISK ASSESSMENT OF UNREINFORCED MASONRY BUILDINGS

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SUMMARY

The scope of the paper is to present the development and application of a new procedure for the seismic risk assessment at urban or territorial scale of unreinforced masonry buildings (*URMB*). The procedure, named *MeBaSe,* is based on four main features: the formulation of the structural capacity and response in terms of mechanics concepts, the representation of seismic demand and structural capacity by means of displacement, the inclusion of the most commonly acknowledged sources of uncertainty and the consideration of in-plane and out-of-plane failure mechanisms. The procedure aims to the evaluation of seismic risk of classes of buildings and is intended as a tool that could in perspective solve most of the drawbacks associated with existing procedures. The uncertainty originating in the material properties and the structural response is treated separately from that coming from the statistics of the population of buildings. The former is directly included in the computation of probability of failure of a class of buildings, whilst the latter is considered in a second stage. The demand, represented by a displacement response spectrum, can be obtained from regional probabilistic seismic hazard studies, and is defined by the median, for each spectral period, and by the corresponding scatter. Concerning the limit states, appropriate median values corresponding to each given drift limit state are used, depending on the type and quality of masonry. For the out-of-plane mechanism, the procedure is restricted so far to simple one-way bending mechanisms. A case study for the validation of the procedure has been carried out in the city of Benevento, Italy, and some preliminary results are presented in the paper, showing the feasibility of the new methodology.

INTRODUCTION

Several methods for seismic vulnerability and risk assessment of existing buildings have been developed in recent years, considering different approaches for the input data and for the results (output). Dolce *et al*. [1] proposed a classification scheme based on the type of input, type of methodology and type of output, according to the options shown in Table 1. It is possible to get different input-method-output combinations, which produce methods with different levels of applicability and accuracy. However, Dolce *et al.* [1] recognized that just hybrid expert-statistical methods were available for masonry

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buildings, and at the time of their publication there was not any method based explicitly on mechanical concepts. Currently, a series of procedures are available for the assessment of *URMB*, with different scopes and capabilities. Each of these procedures shows advantages and drawbacks in their formulation, the latter being usually related with the treatment of the uncertainties that are always unavoidably present in seismic risk estimation. Besides, most if not all the existing methodologies are focused on the determination of the vulnerability of the buildings, leaving for a subsequent stage the computation of the seismic risk, not being clear in some cases the way to achieve this latter objective.

Table 1. Classification of vulnerability assessment methods, after Dolce *et al.* [1]

Input	Method	Output
1. Damage data 2. Geometric and qualitative features 3. Mechanic features 4. Seismic demand features 5. Geological and geotechnical data	1. Statistical methods 2. Mechanical methods 3. Methods based on expert judgements	1. Absolute vulnerability 2. Relative vulnerability

One of the first methods proposed, which has been widely used in Italy, is the so-called damage probability matrix approach (*DPM*), for which several references exist, for example Whitman [2]. The *DPM* is based on the idea that a set of buildings having a common structural typology would have the same behaviour under the seismic action, and consequently, the level of damage would be statistically the same for the given set of buildings. The damage is characterized by some level of uncertainty described by a damage probability matrix. A *DPM* method widely used in Italy is the *GNDT* I level approach [3], having three classes of vulnerability, from A to C, each of these having a *DPM*. For this approach, the demand is considered using the *EMS*-98 intensity scale published in Grüntal [4] and the damage is described by means of a qualitative definition. A clear limitation of this method is the definition of the seismic demand in terms of macroseismic intensity, which has also the additional drawback of being a discrete measure of intensity. The damage probability matrices calibrated on specific building stocks may not be valid in different regions or countries and are affected by some degree of subjectivity which adds uncertainty to the results. A second method widely used in Italy is the *GNDT* II level approach which draws the data from a survey form designed to gather information regarding the typology and constructive features of each single building, that are combined to get an absolute vulnerability index I_V. The vulnerability index is obtained by a weighted average of parameters, of which one only is derived from a mechanical model. The seismic input is represented by *PGA* levels, and the probability of reaching a certain damage level (represented as a scalar D ranging from 0 to 1) is calculated by means of fragility curves in a *D-PGA* domain, using I_V as a parameter.

Probably one of the first uses of a more mechanically-oriented description of the structural behaviour for vulnerability assessment purpose of *URMB* was presented in the *VULNUS* procedure developed by Bernardini *et al*. [5]. This methodology is based on the evaluation of the geometrical and mechanical characteristics of each building, which is combined with the evaluation of some other important factors controlling the response of the structure, that are handled through qualitative judgments. The whole procedure is developed under the fuzzy set theory that is used for the definition of the safety criterion. The methodology has been calibrated for Italian building stocks, and correlation with parameters representing the seismic demand (e.g. *PGA*) for damage prediction is under development.

In recent years further methods have been developed from a different perspective, trying to overcome the limitations in many of the previous approaches. The *HAZUS* methodology was developed by the Federal Emergency Management Agency – *FEMA* [6] and it is based on three fundamental concepts: capacity curve, design point and fragility curve. Capacity curve is the relationship between the lateral load resistance of a given structure and its characteristic lateral displacement, and is obtained by means of a static pushover analysis. The capacity curve is then converted to spectral acceleration and roof displacement in order to be compared with the demand spectrum. The capacity curve is controlled by the yield capacity, or restoring force, and the ultimate capacity, being possible to represent with a single curve the corresponding strength at a certain displacement limit state, which in turn can be directly correlated with some damage limit state. The fragility curve represents the cumulative density function for the probability of reaching or exceeding a specific damage limit state for a given peak response to a given ground motion demand. Potential problems of this methodology are related to the fact that the capacity curves included in the database are strictly valid just for buildings inside the United States, and would require a re-calibration in different environments. In addition to that, for the case of *URMB* the concept of a capacity curve associated to the global response of a building does not seem to take properly into account local or partial out-of-plane mechanisms, which are the most frequent in the most vulnerable buildings.

The *FaMIVE* procedure developed by D'Ayala and Speranza [7] aims to the assessment of the seismic vulnerability of historic buildings in town centres, clearly based on mechanical principles. *FaMIVE* uses a predefined set of rules in the definition of the most feasible collapse mechanism, for which a load factor or collapse multiplier is evaluated by means of a static equivalent procedure. This load factor is further manipulated to lead to the definition of some vulnerability value. Although the method is quite rigorous in the treatment of collapse mechanisms, no clear indication of how to compare the vulnerability index with the seismic demand or hazard is given, and the approach does not tackle the problem of the inherent uncertainty existing in structural capacity, building response and seismic demand. Also, attention is focused on horizontal load capacity, whereas out-of-plane collapse mechanisms were shown to be more susceptible to displacement demand rather than to acceleration, as will be also discussed later.

Recently, other methods have been put forth based on mechanical features, although they are in a preliminary stage or have some aspects worth of being improved. Calvi [8,9] proposed a simplified procedure for the evaluation of the vulnerability of classes of buildings, including a formulation for masonry structures. The proposal by Calvi was one of the first making use of mechanics concepts for the definition of the structural response and the comparison of displacement demand and capacity as a criterion for the evaluation of the vulnerability. However, the method did not include a complete and clear treatment of the uncertainties, and did not account for out-of-plane collapse mechanisms.

A similar approach to that of Calvi has been presently in recent times by Lang [10] based on the displacement evaluation of capacity and demand and considering the evaluation of the in-plane and the out-of-plane failure mechanisms in two separate stages. The method is based on the computation of the displacement capacity of the building, for which a mechanics-based approach is followed, being then the displacement capacity compared with the seismic demand given in the form of a displacement response spectrum. The main drawbacks identified are the definition of the capacity curve and the seismic demand as deterministic relationships, ignoring all the inherent uncertainty existing in their definition, and the applicability of the method is carried out in a building-per-building basis, not being explicitly indicated the possibility of use at urban scale. The method includes basic out-of-plane mechanisms, but again attention is focused on acceleration and horizontal load capacity, rather than displacement demand.

Drawing from the above discussion, the features of an "ideal" methodology could be summarized as follows:

- To use a continuous rather than a discrete measure of the earthquake demand, and to use a more complete representation of the demand in the form of response spectra, rather than single parameters such as *PGA*.

- To use displacement or deformation as an indicator of the demand level, which has a better correlation with the level of damage.
- To consider the uncertainty on the seismic demand, recognized to be one of the components of uncertainty that are required to be included in a complete seismic risk assessment.
- To consider the sources of uncertainty coming from capacity and response.
- To use mechanical criteria for the definition of the structural capacity, including the most important parameters affecting masonry strength, for example shear strength, friction, etc.
- To include in the response the most common out-of-plane failure mechanisms.
- To include the foundation and non-structural components in the assessment.
- To take into account the experience obtained from past earthquakes that have occurred in similar conditions, in order to improve the parameters and structural models used in the analysis.
- To have the possibility of evaluating the seismic vulnerability of a given class of buildings, considering different levels of quality and completeness in the data and in the refinement in modelling and analysis.
- To be of simple and fast application to real cases, avoiding to the maximum extent the use of expensive and time consuming computational tools.
- To avoid, when possible, subjectivities in the definition of parameters, factors and relative weights.
- To require minimal adjustments for its applicability in different regions of the world.
- To be coded as stand-alone software, not strictly linked to any specific proprietary software.

The procedure presented in the following sections is a possible first step towards the fulfilment of this list of desirable features.

MECHANICAL MODELS

The proposed *Me*chanics-*Ba*sed *Se*ismic risk assessment method (*MeBaSe*) stems from the idea of the displacement-based approach for vulnerability evaluation of classes of buildings proposed by Calvi [8,9].

Simplified mechanical model for in-plane failure modes

The initial idea is the modelling of a multi-degree of freedom system (*MDOF*) by means of a single-degree-of-freedom (*SDOF*) substitute-structure using the method proposed by Shibata and Sozen [11]. Figure 1 shows the *MDOF* defined by the masses m_{fi} located at each storey height h_i and subjected to a system of lateral forces F_i. This system is represented by an equivalent *SDOF* system with effective mass m_{eff} and effective stiffness K_{eff}, given by the ratio between the yielding force of the system F_y and the effective displacement Δ_{eff} for a given displacement demand or limit state, Δ_{LS}.

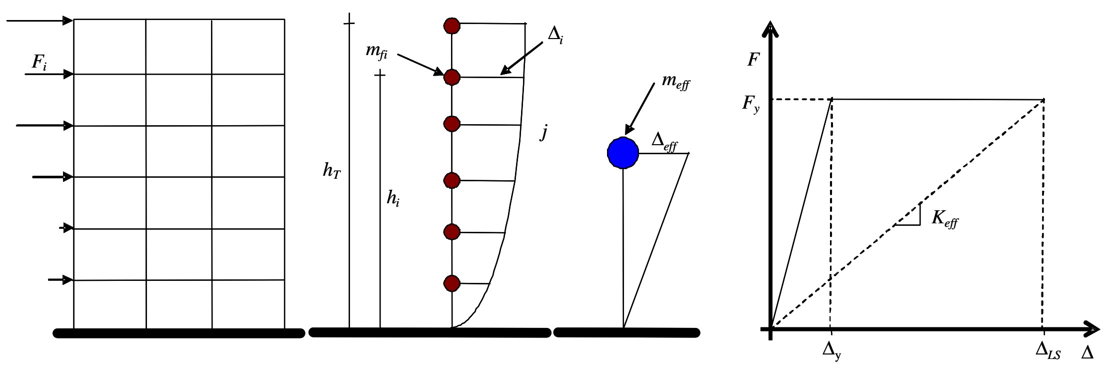

Figure 1. Simplified model for an equivalent single degree of freedom system

This simplification in modelling must account for the different displacement profiles, according to the failure mechanism or displacement profile at a given limit state. In Figure 2 four possible displacement profiles for different limit states and in-plane failure modes are shown. Profile (a) corresponds to the limit states *LS1* (no damage) and *LS2* (minor structural damage) for which none of the members of the structure has reached the yield displacement Δ_y (i.e. an appreciable extension of cracking). For the yield displacement profile a triangular displacement shape is assumed, which is valid for low-rise buildings dominated by a shear-deformation failure mode, as is the case for most old-unreinforced masonry buildings. The profiles (b), (c) and (d) correspond to the limit states *LS3* (significant structural damage) and *LS4* (collapse), in which either a soft storey mechanism or a weak beam mechanism has been developed as the most probable failure mode. Profile (b) is the most typical situation for low rise unreinforced masonry buildings, but profile (c) could be also possible, either at top or intermediate stories, depending on the relative strengths of the stories; or also profile (d), depending on the relative strengths of spandrel beams and piers.

The maximum displacement for a given limit state Δ_{LS}, can be computed as the summation of the yield displacement Δ_y and the plastic displacement Δ_p with Eq.(1), where h_{sp} is the effective height of the piers going into the inelastic range, when openings are present, or the storey height when there are no openings. For a regular distribution of masses and a uniform storey height κ_1 is equal to 0.667, but when the mass distribution is not regular, like is common in the case of *URMB*, the buildings are better represented by a distributed mass corresponding to the mass of the masonry walls plus lumped masses corresponding to the mass at each floor. In this case, κ_1 and κ_2 have to account for the different mass distribution. A detailed computation of these two factors is extensively explained in Restrepo-Vélez [12] and the proposed values are presented in Table 2, being considered as valid when the weak storey is formed in the two lower thirds of the building.

$$\Delta_{LS} = \kappa_1 h_T \delta_y + \kappa_2 \left(\delta_{LS} - \delta_y\right) h_{sp} \quad (1)$$

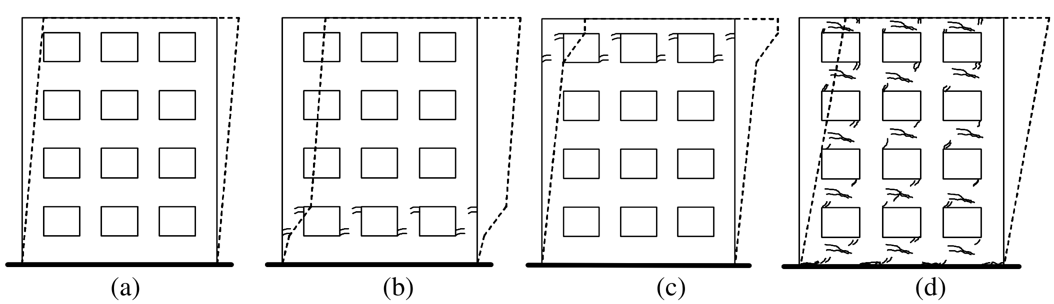

Figure 2. Deformed shapes for different limit states and in-plane failure modes

Table 2. Values of κ_1 and κ_2 for each number of stories

Number of stories *n*	κ_1	κ_2
1	0.790	0.967
2	0.718	0.950
3	0.698	0.918
4	0.689	0.916
5	0.684	0.900
6	0.681	0.881

With the effective stiffness, computed as the secant stiffness at a given limit state K_{eff}, the effective period at the same limit state T_{LS} is obtained with Eq.(2), being C_m the implicit seismic coefficient for every storey, and the minimum C_m value will be the one controlling the overall strength of the structure, assuming elasto-plastic behaviour.

$$T_{LS} = 2\pi \sqrt{\frac{\Delta_{LS}}{\frac{1}{\phi_c}\min\{C_m\}g}} \quad (2)$$

A possible definition of C_m considers just the shear capacity of the walls as the only source of lateral strength and is computed with Eq.(3) as proposed by Benedetti and Petrini [13]. In this equation A_m is the normal area of all the resistant walls in direction of minimal strength, τ_{km} is the referential shear strength for masonry at the level m, n is the number of floors, and γ_m is the ratio between A_m and the maximum area B_m of the longitudinal or transversal walls.

$$C_m = \frac{1}{K_1} A_m \tau_{km} \left[1 + \frac{K_2}{\tau_{km} A_m (1+\gamma_m)}\right]^{\frac{1}{2}} \quad (3)$$

Eq.(2) is explicitly written in Eq.(4) including all the parameters influencing the value of the effective period plus an extra ε_T term representing the implicit uncertainty existing in the use of this expression. Besides, and explicit inversion of Eq.(2) is shown as Eq.(5), showing in a convenient way the dependence of displacement on period.

$$T_{LS} = 2\pi \sqrt{\frac{\kappa_1 h_T \delta_y + \kappa_2 (\delta_{LS} - \delta_y) h_{sp}}{\frac{1}{\phi_c} A_m K_1 \tau_{km} \left(1 + \frac{K_2}{A_m (1+\gamma_m)\tau_{km}}\right)^{1/2} g}} + \varepsilon_{T_{LS}} \quad (4)$$

$$\Delta_{LS} = \frac{T_{LS}^2}{4\pi^2} \frac{1}{\phi_c} A_m K_1 \tau_{km} \left(1 + \frac{K_2}{A_m (1+\gamma_m)\tau_{km}}\right)^{1/2} g + \varepsilon_{\Delta_{LS}} \quad (5)$$

K_1 and K_2 are obtained with Eq.(6) and (7) respectively.

$$K_1 = \frac{1}{W_t \sum_{i=m}^{n} \frac{h_i W_i}{\sum_{j=1}^{n} h_j W_j}} \quad (6) \qquad K_2 = \frac{\sum_{i=m}^{n} W_i}{1.5} \quad (7)$$

A correction coefficient ϕ_c has been included in Eq.(2) in order to cope with possible three-dimensional effects and other sources of strength not included in the formulation. To define ϕ_c different building configurations, from two to five storeys, were analyzed with this simplified approach and by using the three-dimensional non-linear computer code *SAM*, developed at the University of Pavia by Magenes *et al.* [14]. The buildings were selected trying to cover a realistic range of number of storeys, structural configurations and lateral strengths. A general description of each building can be found in Restrepo-Vélez [12]. As an example of the analyses Figure 3 shows the pushover results for a four-storey building obtained with the simplified and the three-dimensional methodologies, and for different values of τ_{km}. The results show that for the simplified analyses the lateral capacity of the building increases as the referential shear strength increases, as it would be expected. The results for the three-dimensional analyses show that for the higher values of referential shear strength, the maximum lateral strength force is virtually the same, decreasing just for the lowest value of τ_{km}. This behaviour is explained considering that for high values of τ_{km} the walls and spandrel beams have a shear strength higher than the flexural strength, so the failure mechanisms are dominated by flexural-type modes, being insensitive to the variation of τ_{km}, whilst for low values of τ_{km} the controlling modes are the shear-type. It is interesting to note also that for high values of τ_{km} the simplified methodology gives higher lateral strength than the three-dimensional methodology, whereas for low to medium values of τ_{km} both results are closer and for the lowest values of τ_{km} the simplified results are below the three-dimensional results.

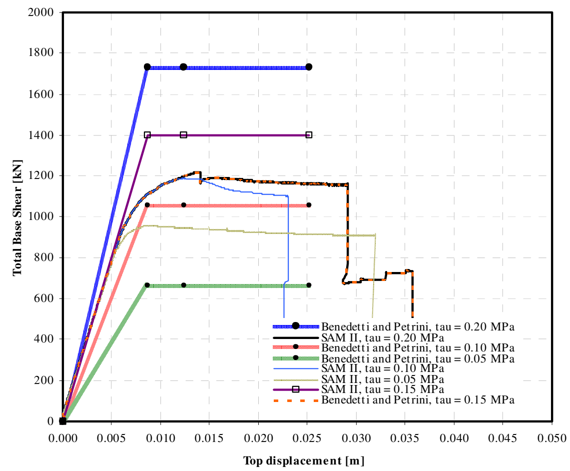

Figure 3. Pushover results for a four-storey building with the simplified and three-dimensional methodologies, and for different referential shear strength values

It was assumed that the correction factor must consider the variation of τ_{km} and some parameter to include the susceptibility of any given building to fail in flexural or shear-dominated types of collapse. In Restrepo-Vélez and Magenes [15] it is proposed as a proper indicator of this susceptibility the ratio of the length of the resisting piers L_W to the total length of walls, including openings and non-structural walls L_T. The higher this ratio is the more likely is to have shear type of failure on the walls, whilst for low values of this ratio it would be more likely to have failure mechanism dominated for the flexural collapse of the spandrel beams and concrete ring beams. From the point of view of the gathering of field information the selected parameters have the property of being easily measured or reliably estimated during a survey. The correction factor is defined as the ratio between the maximum lateral force computed with the simplified methodology and the maximum lateral force computed with the three-dimensional methodology.

A regression analysis was carried out to find a relationship between τ_{km} /(L_W/L_T) and ϕ_c, as shown in Figure 4 and Eq.(9). Details on the evaluation of the ϕ_c are presented in Restrepo-Vélez [12]. ε_c represents the scatter on the relationship, whose magnitude was estimated through the computation of new factors ϕ_c^* and then the corresponding residuals ρ were obtained with Eq.(10):

$$\phi_c = 5.53 \frac{\tau_{km}}{\left(L_W / L_T \right)} + 0.46 + \varepsilon_c \tag{9}$$

$$\rho = \phi_c - \phi_c^* \tag{10}$$

The suitability of the normal distribution as a reasonable model to describe the probability distribution of the residuals ρ was then verified. Figure 5 shows the cumulative density function (*CDF*) and the mass Density Function (*MDF*) for the residuals ρ. This additional source of variability is also incorporated into the analysis, as will be explained later.

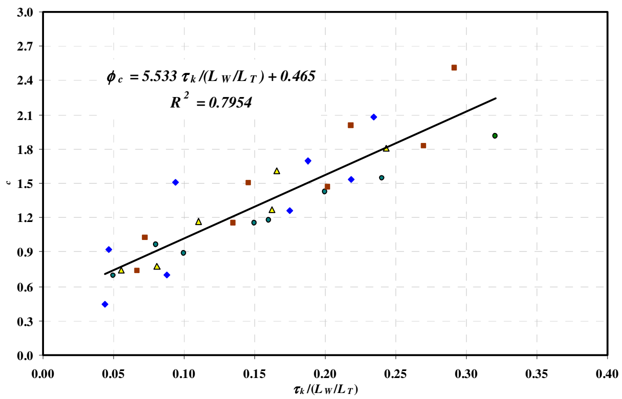

Figure 4. Empirical relationship between $\tau_{km}/(L_W/L_T)$ and ϕ_c

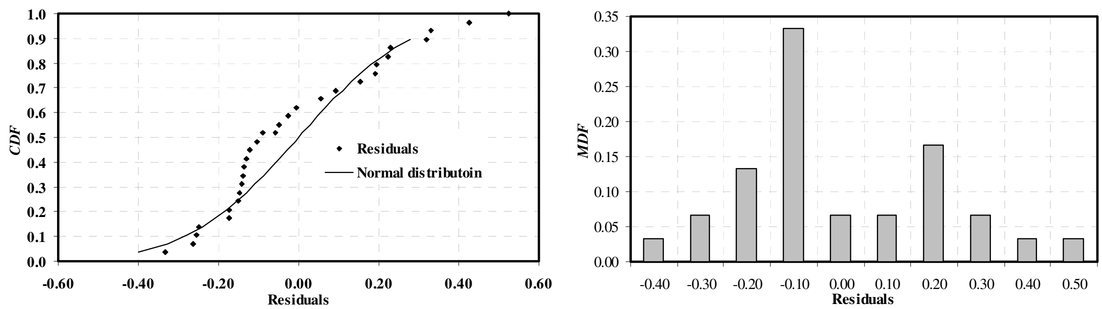

Figure 5. Cumulative density function (*CDF*) and Mass Density Function (*MDF*) for residuals ρ

Simplified mechanical model for out-of-plane failure modes

The procedure developed so far is restricted to simple configurations like those presented in Figure 6. The definition of adequate limit states for the assessment of out-of-plane unreinforced masonry walls is based on the work by Doherty *et al*. [16] and Griffith *et al*. [17], which have represented the semi-rigid nonlinear behaviour of a bending wall by means of the tri-linear simplified model shown in Figure 7. F_o corresponds to the force at incipient rocking or "Rigid threshold", Δ_u is the displacement at the point of static instability. The straight line defined by these two parameters means that the wall is cracked before the motion starts, and that the wall behaves as a set of rigid bodies. It was shown that the nonlinear nature of the masonry walls, subjected to out-of-plane motion, can be described by the tri-linear model, defined by the parameters Δ_1 and Δ_2, these parameters being estimated from the material properties and the state of degradation of the cracked section at the pivot points, as a proportion of the ultimate displacement, as shown in Table 3. Griffith *et al*. [17] showed that the displacement Δ_2 is the most relevant parameter, when the main objective is to determine whether a wall will collapse or not. For the definition of limit state, "stable" or "collapsed" are currently the only possibilities considered in the method. The ultimate displacement or the displacement at the point of instability is a function of the thickness of the wall, the boundary conditions and Ψ, which is the ratio between the axial force applied at the top the wall and the self-weight of the upper-half of the wall above mid-height. F_o is the force required to initiate the rocking motion, and is a function of the overburden ratio Ψ, the effective mass and the geometry of the wall, and can be expressed according to Eq.(11).

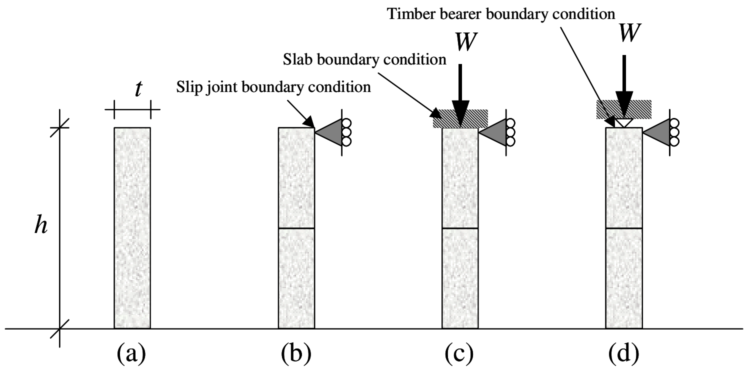

Figure 6. Boundary conditions considered

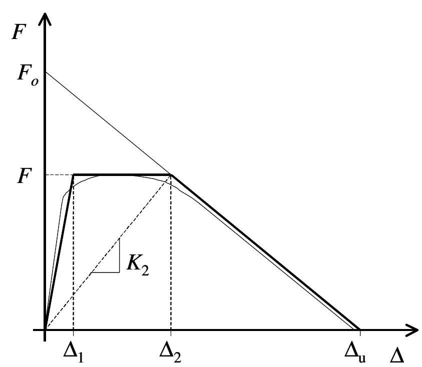

Figure 7. Tri-linear simplified model

Table 3 Displacement ratios for the tri-linear model

State of degradation at cracked joint	Δ_1 / Δ_u	Δ_2 / Δ_u
New	0.06	0.28
Moderate	0.13	0.40
Severe	0.20	0.50

$$F_o = \frac{4(1+\Psi)M_e g t}{h} \tag{11}$$

Considering the relationship between stiffness and period, it is possible to get an expression to link the period T_2 and the limiting displacement Δ_2, as is presented in Eq.(12). Here the limiting displacement is conservatively assumed as $\Delta_{LS} = \phi\Delta_u$, being ϕ a factor between 0.8 and 1.0, and $\rho_2 = \Delta_2/\Delta_u$. The corresponding expression for displacement is Eq.(13).

$$T_{LS} = \left(\frac{\pi^2 \rho_2 \phi \Delta_u h}{\phi(1-\rho_2)(1+\Psi)gt}\right)^{1/2} + \varepsilon_{T_{LS}} \tag{12}$$

$$\Delta_{LS} = T_{LS_i}^2 \frac{\phi(1-\rho_2)(1+\Psi)gt}{\pi^2 \rho_2 h} + \varepsilon_{\Delta_{LS}} \tag{13}$$

In Figure 8 the most common out-of-plane failure mechanisms are presented. Mechanisms from A to G are taken from D'Ayala and Speranza [7]. Mechanisms *H* are relevant to double-leaf walls. Eq.(13) and (14) are applicable to mechanisms *A*, *E, F,* and *H*. Extension of these ideas for the remaining mechanisms is currently under way, assuming that all the mechanisms being studied can be represented by a tri-linear or an equivalent model.

SEISMIC DEMAND

In the case of in-plane mechanisms, the seismic demand is represented by the elastic displacement response spectrum, properly scaled to consider the effective damping related to each limit state. The demand is defined by the median or mean value for each spectral period and by the standard deviation of the corresponding demand parameter. In the *MeBaSe* procedure, it is proposed that for a scenario event

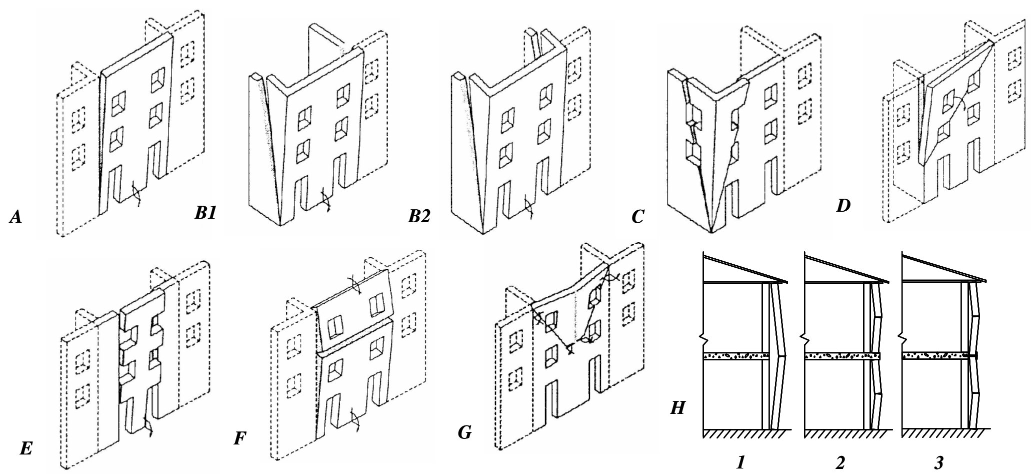

Figure 8. Out-of-plane failure mechanisms

the *CDF* of demand will be computed from the median values of the displacement response spectrum given by the probabilistic seismic hazard analysis, and the standard deviation representing the epistemic uncertainty in the definition of the spectrum. The aleatory component of the uncertainty on the demand is assumed included when the probabilistic seismic hazard assessment is carried out. A discussion of this issue is presented in Restrepo-Vélez [12] and Restrepo-Vélez and Magenes [15].

Similar ideas apply in the case of the out-of-plane mechanisms. Besides, in this paper it is assumed that the equations proposed in Eurocode 8 [18] to estimate the acceleration demand for non-structural elements can also be used for out-of-plane walls, being converted to displacement by means of Eq.(14), where: α is equal to the peak ground acceleration of the given ground – motion, h_{ne} is the height at which the centroid of the element is placed, h_T is the total height of the building, T_{ne} is the period of vibration of the element and T_1 is the period of the first mode of vibration of the building.

$$S_{d_{ne}} = \frac{T_{ne}^2}{4\pi^2} \frac{3\alpha\left(1 + {}^{h_{ne}}\!/_{h_T}\right)}{1 + \left(1 - {}^{T_{ne}}\!/_{T_1}\right)^2} \tag{14}$$

PROBABILISTIC ASPECTS

Sources of uncertainty

It has been widely accepted that in seismic risk estimation the uncertainty is originated in three different sources, namely: ground-motion demand, damage state threshold and capacity response. The uncertainty on the ground-motion demand is readily accounted in the procedure, as will be shown in the next section. The uncertainty coming from the damage state threshold and the capacity response are properly considered with the developing of a suitable joint probability density function (*JPDF*) for T_{LS} and Δ_{LS}, which is presented next. A comprehensive description of the different types of uncertainties is presented by Abrahamson [19] and Restrepo-Vélez [12]

Definition of a suitable *JPDF* for Structural Capacity

The definition of the *JPDF* $f_{\Delta,T}(\Delta,T)$ is a requirement for the formulation of risk estimation presented in this paper. Several reliability techniques can be used to define the *JPDF*, which should include the effect of all the related sources of uncertainty and that cannot be estimated from the sample with enough accuracy. In the classical reliability formulation presented in Eq.(15) $F_D(\alpha)$ represents the *CDF* of the demand being less or equal to some value α, and $f_{SC}(\alpha)$ is the *PDF* of the structural capacity *SC*:

$$P_f = \int_0^{\infty} [1 - F_D(\alpha)] f_{SC}(\alpha) d\alpha \qquad (15)$$

In the particular formulation representing the capacity for a given limit state *LS* in terms of the displacement capacity for that given limit state Δ_{LS} and the corresponding effective period T_{LS}, the *JPDF* $f_{\Delta_{LS} T_{LS}}$ is expressed as the product of the *PDF* of Δ_{LS} conditional to some T_{LS} times the *PDF* of T_{LS}, leading to the computation of the probability of failure P_f with Eq.(16).

$$P_f = \int_y \int_x [1 - F_D(\Delta_{LS} = x / T_{LS} = y)] f_{\Delta_{LS}/T_{LS}}(x/y) f_{T_{LS}}(y) dx dy \qquad (16)$$

The first order reliability method (*FORM*) is used to compute the required *CDF*s, from which the corresponding *PDF*s are numerically evaluated. *FORM* is a simpler and faster alternative compared with standard Monte Carlo simulation methods, and even with improved versions like Importance Sampling or Latin Hypercube Sampling. Additionally, in the case of this particular application *FORM* method has proved to be accurate enough, as can be seen in Figure 9, in which the *CDF*s for T_{LS} estimated with *FORM* and with Importance Sampling methods are compared, showing a good agreement.

Risk assessment at urban scale

A key issue in the risk assessment at urban scale is the definition of the building classes. *MeBaSe* allows virtually any criterion in the definition of building classes. For example, Figure 10 shows a hypothetic region that has been subdivided in n_1 zones. The building stock within the particular $zone_i$ (i = 1 to n_1) is subdivided in building classes $CB_j(mt,\#s)$ (j = 1 to n_2), according to the material (masonry type, age, quality) *mt*, and number of stories *#s*. Afterwards, the probability of reaching or exceeding the specified limit state of interest $P_{f_{ij}}$, for each $CB_j(mt,\#s)$ in each $zone_i$ is computed. The probability of failure in a given zone for a specific class of buildings $CB_j(mt,\#s)$ is computed with Eq.(17), $P_{mt,\#s}$ is the joint probability that a building within the population has a given material *mt* and certain number of stories *#s*, which can be statistically inferred from the sample gathered during the data survey. The total probability of failure in the whole region for the same class of buildings $CB_j(mt,\#s)$ is computed with Eq.(18).

$$P_{f_{CB_j(mt,\#s)}} = P_{f_{ij}} P_{mt,\#s} \qquad (17)$$

$$P_{f_{CB_j(mt,\#s)_T}} = \sum_{i=1}^{n_1} P_{f_{CB_j(mt,\#s)_i}} \qquad (18)$$

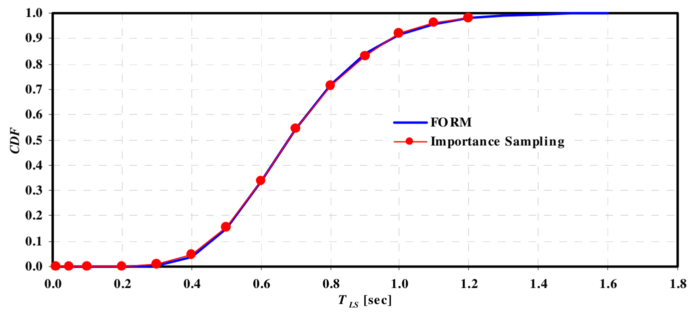

Figure 9. Comparison of the *CDF* for T_{LS} obtained with *FORM* and Importance Sampling

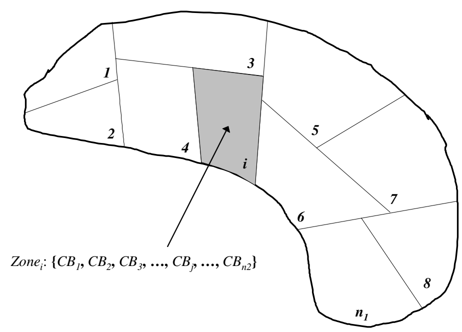

Figure 10. Subdivision of the hypothetic region in zones

APPLICATION OF THE PROCEDURE IN BENEVENTO - ITALY

The following preliminary results correspond to an application of the *MeBaSe* procedure to the *URMB* stock in the Rione Libertà at Benevento - Italy. Initially, a survey was carried out with the help of a survey form specifically designed for this application (Restrepo-Vélez [12]). Figure 12 shows in cyan, yellow and red the location of all the identified *URMB* in this neighbourhood. Each colour corresponds to a class of building, defined according to the number of stories *#s* and the quality of the masonry *mt*, that here is basically represented by an scalar corresponding to certain range of values for the parameter τ_{km}. The geometrical and material parameters gathered during the survey were used to compute the mean and standard deviation of the parameters of interest, for classes of buildings 3 – 4, 4 – 4 and 5 – 4, of which typical buildings are shown in Figure 13.

Table 4 shows the mean and standard deviation for each one of the eleven parameters used in the definition of T_{LS} and Δ_{LS} for in-plane mechanisms, according to Eqs. (4), (5) and (16). In the case of the parameters for the drift limit at yield δ_{LS} and the drift limit at limit states δ_{LS} *LS1* and *LS4*, the values were taken from Soresi [20], corresponding to tuff, which is the type of stone used in the construction of all the surveyed buildings at Benevento. For this specific example, the scatter represented by $X_{\varepsilon_{T_{LS}}}$ and $X_{\varepsilon_{\Delta_{LS}}}$ has been neglected. The $P_{f_{ij}}$ for classes 3 – 4, 4 – 4 and 5 – 4 corresponding to *LS1* and *LS4*, and to ground-motion levels of 0.10g and 0.35g, are presented in Table 5. Finally, considering that the probability that a building in Rione Libertà belongs to certain class can be estimated as 39/60, 20/60 and 1/60, for classes 3 – 4, 4 – 4 and 5 – 4, respectively, the total probabilities of reaching or exceeding a limit state would be those indicated in Table 6. The procedure is also presently being applied to the historical centre of Benevento, considering different typologies of buildings.

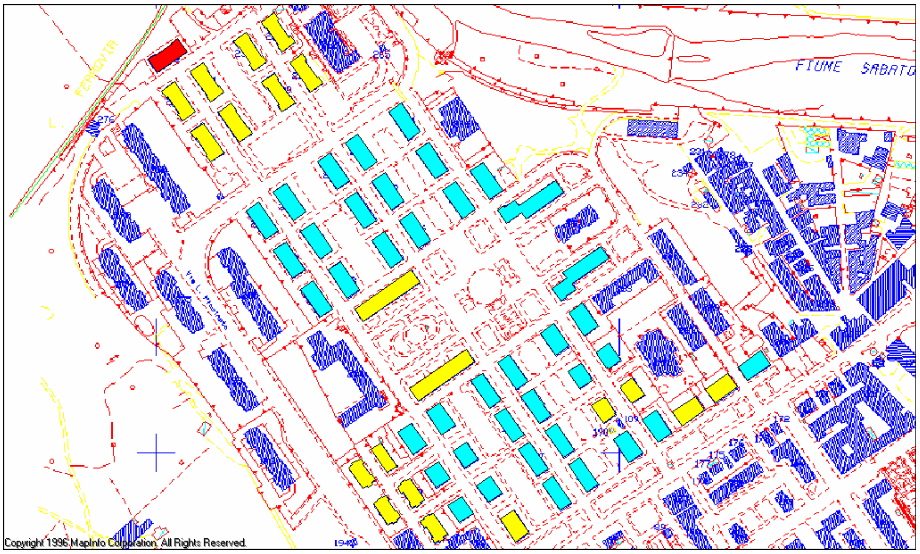

Figure 12. Classes of buildings according to number of stories and quality of masonry

Figure 13. Typical buildings for classes 3 – 4, 4 – 4 and 5 – 4

Table 4. Mean and standard deviation values for vector *X* for *LS1* and *LS4*

	Class 3 – 4		Class 4 – 4		Class 5 – 4	
Parameter	μ_x	σ_x	μ_x	σ_x	μ_x	σ_x
h_T [m]	10.7	0.13	12.8	1.71	14.4	1.50
h_{sp} [m]	2.1	0.11	1.9	0.29	2.0	0.30
δ_y	0.0013	0.00046	0.0013	0.00046	0.0013	0.00046
δ_{LS1-2}	0.0013	0.00046	0.0013	0.00046	0.0013	0.00046
δ_{LS3}	0.0034	0.00110	0.0034	0.00110	0.0034	0.00110
δ_{LS4}	0.0045	0.00135	0.0045	0.00135	0.0045	0.00135
τ_{km} [MPa]	0.087	0.051	0.087	0.051	0.087	0.051
L_w/L_T	0.629	0.142	0.809	0.148	0.660	0.100
X_ε	1.030	0.254	1.030	0.254	1.030	0.254
K_1[1/kN]	0.108	0.021	0.069	0.031	0.050	0.020
A_m[m^2]	11.205	6.460	19.520	11.700	10.400	2.000
K_2[kN]	4.133	2.076	9.299	4.794	10.600	3.500
γ_m	1.463	0.336	1.496	0.193	2.500	0.200

Table 5. Probability of reaching or exceeding a limit state, for different classes of buildings

Class	*LS1* – 0.10g	*LS1* – 0.35g	*LS4* – 0.10g	*LS4* – 0.35g
3 – 4	0.461	0.920	0.267	0.872
4 – 4	0.380	0.841	0.227	0.772
5 – 4	0.549	0.872	0.350	0.853

Table 6. Total probabilities of reaching or exceeding a limit state, for different classes of buildings

Class	*LS1* – 0.10g	*LS1* – 0.35g	*LS4* – 0.10g	*LS4* – 0.35g
3 – 4	0.299	0.598	0.174	0.567
4 – 4	0.127	0.280	0.076	0.257
5 – 4	0.009	0.015	0.006	0.014

CONCLUSIONS

A simplified mechanics-based procedure for the seismic risk assessment of classes of *URMB* has been presented. The *MeBaSe* procedure complies with the advantages and solves, at least partially, the disadvantages identified in existing procedures. The method makes use of displacement to represent the seismic demand, considers the uncertainties on seismic demand, structural capacity and dynamic response. It uses mechanical criteria for the definition of the structural capacity and allows the inclusion of the most common out-of-plane failure mechanisms. The information obtained from past earthquakes can be used to improve the definition of the parameters and structural models. The method also can consider different levels of quality and completeness in the data and in the refinement in modelling and analysis. As currently defined, it does not require expensive and time consuming computational tools, and has been coded as a stand-alone software, totally affordable to be implemented.

Currently, a research project is being carried out to test the method on a typical *URMB* stock in Colombia, which would possibly confirm the applicability of the methodology in different countries.

Should the method be used in a building stock whose structural properties differ from the typical building stock in Italy, some factors would require to be verified or adjusted. For what concerns in-plane mechanisms the factors to be verified are the correction factor ϕ_c and the range of possible values of τ_{km} and δ_{LS} according to the types of masonry under study. Regarding out-of-plane mechanisms, the factors to be validated for the simple configurations included so far in the method are the displacement ratios Δ_1/Δ_u and Δ_2/Δ_u. The value of standard deviation used for any of the parameters must reflect the specific level of uncertainty for each application.

Additional research is still required to define the influence of the type of distribution selected for each random variable in the estimation of the *CDF*s for T_{LS} and Δ_{LS}. Furthermore, the effects of the duration of seismic demand and the variability of the scaling factors to modify the demand spectrum according to the level of apparent ductility for each limit state also need further investigation.

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