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ISSUES ON THE USE OF TIME-HISTORY ANALYSIS FOR THE DESIGN AND ASSESSMENT OF MASONRY STRUCTURES

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Abstract. *Nonlinear dynamic analysis is widely recognized as the most accurate analysis technique for the design and assessment of structures. However, its use is still not common in the everyday engineering practice, mainly due to various issues concerning its performance and the interpretation of its results. A first issue includes the limited availability of computer programs that allow the performance of time history analysis, especially for the case of masonry structures, although some software are currently available to this aim. Another difficulty is related with the selection of appropriate input ground motion records to be used for the analysis. Real records are well known to be a preferable choice with respect to artificial or synthetic ground motions, but the limited availability of real records often requires scaling them, with all the concerns associated with this operation. Also, a proper selection of seismic input requires some level of expertise, which is not so common in the professional field. A third problem regards the difficulty in the interpretation of the results of nonlinear dynamic analysis in terms of performance limits. The definition of significant limit states in relation to the results of nonlinear dynamic analyses is still indeed a very open problem and it is related to the fact that the achievement of a local limit condition (e.g. failure of a pier element) would not adequately represent the overall damage state of the building. Therefore there is the need to find a definition of limit states describing the global building performance, i.e. taking into account not only the peak concentrated damage in a single element, but also the diffusion of damage through the different structural elements and the evolution of the global collapse mechanism. This would also allow for a rational implementation of the qualitative definition of damage states commonly adopted in performance-based earthquake engineering.*

1 INTRODUCTION

As reported in EC8 (clause 4.3.3.1(5) of EN 1998-1 [1]) "Non-linear analyses should be properly substantiated with respect to the seismic input, the constitutive model used, the method of interpreting the results of the analysis and the requirements to be met."

This basic statement well identifies the main issues related to the use of nonlinear analysis and, in particular, of nonlinear time-history analysis, which is indubitably the most accurate method for assessing the seismic response of structures provided that these critical issues are properly tackled and suitable tools are used.

Nonlinear dynamic analysis requires the seismic input to be represented in terms of properly defined time-series (e.g. accelerograms), which need to be consistent with the seismic hazard at the site. In many building codes, this idea is associated with the concept of "spectrum-compatibility", that will be discussed in more detail in section 3.

A suitable modeling strategy for the analysis of the dynamic seismic response of complete masonry buildings is presented and discussed in section 2.

Finally, an extended discussion on the definition and identification of appropriate limit states for the interpretation of the results of nonlinear time-history analysis of masonry buildings is presented in section 4. Different criteria are compared and some suggestions are given based on their application to five building models.

2 A NONLINEAR MODEL FOR DYNAMIC ANALYSIS OF URM STRUCTURES

The need for nonlinear analysis tools for complete masonry buildings arose in the late 1970s in Italy and in Slovenia, where simplified modeling techniques and analysis methods were developed and adopted in practice [2]. In the following decades several other nonlinear models were developed and some of them are already available to practitioners and make now possible to carry out reliable nonlinear pushover analysis of masonry structures [3][4]. These methods, generally based on the equivalent frame approach [5][6][7] and the macro-element discretization (single 2-node elements modeling structural members such as piers and spandrel beams), require a limited computational burden since the number of degrees of freedom and elements in the structural model is limited.

An effective equivalent-frame formulation allowing the dynamic global analysis of whole buildings, when only in-plane response of walls is considered, is available in the TREMURI model [8][9].

The nonlinear macro-element model representative of a whole masonry panel described in Penna et al. [10] permits, with a limited number of degrees of freedom (8), to represent the two main in-plane masonry failure modes, i.e. bending-rocking and shear-sliding (with friction) mechanisms (and their interaction), on the basis of mechanical assumptions. This model was explicitly formulated [11] to simulate the cyclic behavior of masonry piers, considering, by means of internal variables, the shear damage evolution, which controls the strength deterioration (softening) and the stiffness degradation. The macro-element also accounts for the effect (especially in bending-rocking mechanisms) of the limited compressive strength of masonry: toe crushing effect is modeled by means of a phenomenological non-linear constitutive law with stiffness degradation in compression. Recent developments [12] have also extended the macro-element capabilities including second order effects which can be important in case of large displacements or for other applications of the model (e.g. simulation of local/out-of-plane failure modes).

A frame-type representation of the in-plane behavior of masonry walls is adopted: each wall of the building is subdivided into piers and spandrel beams (2-node macro-elements) connected by rigid areas (nodes). Post-earthquake damage observation shows, in fact, that only rarely (very irregular geometry or very small openings) cracks appear in these areas of the wall. Hence, the deformation of these regions is assumed to be negligible, relatively to the macro-element non-linear deformations governing the seismic response. The presence of ring beams, tie-rods (non-compressive truss elements), previous damage, heterogeneous masonry portions, gaps and irregularities can be easily included in the structural model (Figure 1).

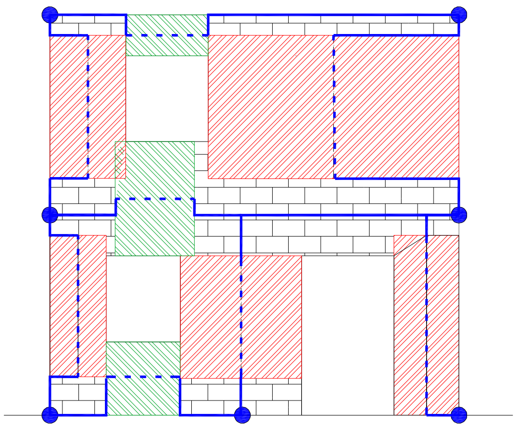


Figure 1 Example of macro-element modeling of a masonry wall (piers in red and spandrels in green).

The presence of reinforced concrete elements (e.g. ring beams, internal columns and walls in mixed structures) can be also modeled by 2-node elements with simplified Takeda nonlinear behavior (Figure 2).

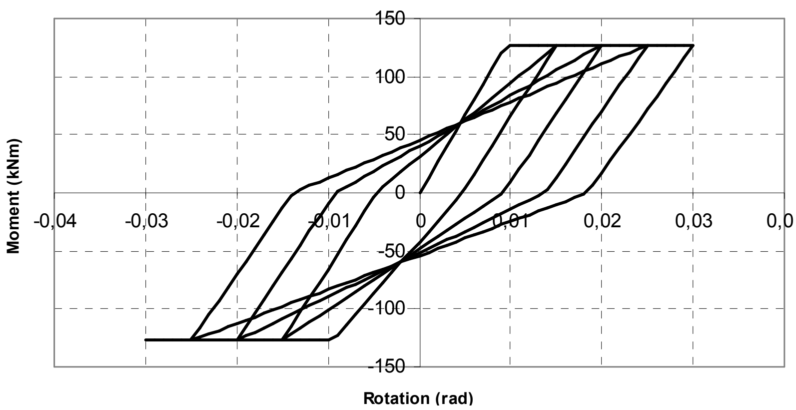


Figure 2 Bilinear simplified Takeda model for simulating the cyclic response of RC members [13].

As described in [9], three-dimensional modeling of whole URM buildings starts from some hypotheses on their structural and seismic behavior: the bearing structure, both referring to vertical and horizontal loads, is identified, inside the construction, with walls and floors (or vaults); the walls are the bearing elements, while the floors, apart from sharing vertical loads to the walls, are considered as planar stiffening elements (orthotropic 3-4 nodes membrane elements) governing the distribution of the horizontal actions between the walls. The local flexural behavior of the floors and the wall out-of-plane response are not computed because they are considered negligible with respect to the global building response, which is governed by their in-plane behavior (a global seismic response is possible only if vertical and horizontal elements are properly connected).

In order to perform non-linear seismic analyses of URM buildings a set of analysis procedures has been implemented: incremental static (Newton-Raphson) with force or displacement control, 3D pushover analysis with fixed and adaptive load pattern ([14]) as well as 3D time-history dynamic analysis (Newmark integration method; Rayleigh viscous damping).

The results of the simulation of the response of the quasi-static tests performed on a full-scale two-story clay brick masonry building ([15]) reported in Figure 3 show the capability of the equivalent-frame macro-element model in reproducing the experimental hysteretic behavior [4].

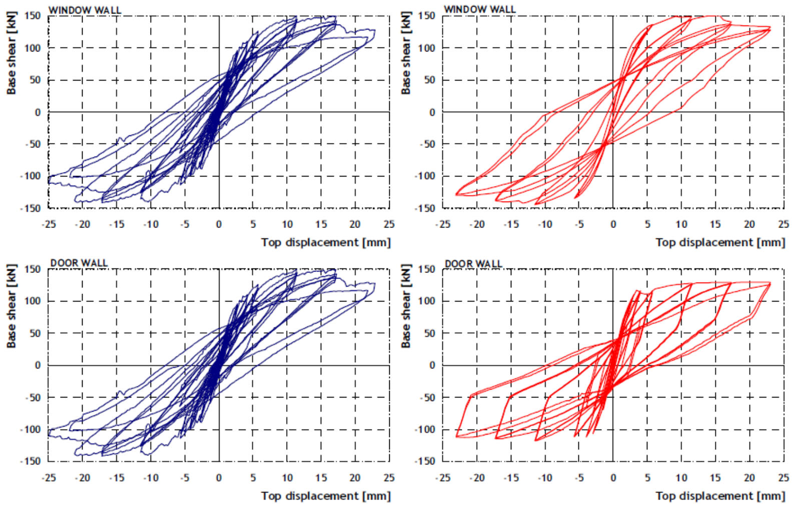


Figure 3 Comparison of experimental (left) and numerical (right) force-displacement curves for the two main walls of the clay brick masonry building tested by [15].

3 SELECTION OF INPUT GROUND MOTIONS FOR TIME HISTORY ANALYSIS

As already mentioned, the execution of time history analyses requires the definition of the seismic action in terms of appropriately selected time-series.

As discussed in more detail in other works (e.g. [18],[19]), accelerograms are typically subdivided in three categories: real (or natural) records selected from accredited strong-motion databases, synthetic accelerograms generated through complex mathematical models of the seismic source and wave propagation phenomena, and artificial accelerograms generated by stochastic algorithms and constrained to be spectrum-compatible to a target response spectrum. Although the choice of the type of record to be used for defining the seismic input for time history analyses depends on the problem under study, in many cases real accelerograms are the best choice, since they are more realistic than spectrum-compatible artificial records and easier to obtain than synthetic seismograms generated from seismological source models. Since they are genuine records of ground shaking produced by real earthquakes, they retain all the ground motion characteristics (e.g. amplitude, frequency, energy content, duration, number of cycles, and phase) and reflect all the factors that influence the seismic motion (i.e., source, path, and site). Moreover they correctly reflect the correlation between the vertical and horizontal components of motion.

As the definition of seismic hazard at the site is usually performed in probabilistic terms, which also account for maximum effects potentially caused by different events, the selection of real records compatible with the expected seismic demand, usually represented in terms of response spectra, necessarily requires the selection of multiple records. Each selected record contributes in a different way to this envisaged compatibility. It is then not surprising that a significant record-to-record variability is commonly found in the selected sets and that it can be particularly relevant in case of nonlinear analysis of degrading systems like masonry structures. Hence, the outcome of the analysis implies a dispersion of the results, normally increasing as the nonlinear component of the structural response increases. This dispersion in the assessed response has to be properly coped with when interpreting the analysis results, as discussed in the following sections.

As also required by several building codes, the consistency of the selected time-series with the seismic hazard is often associated with the idea of “spectrum-compatibility”, normally consisting in imposing that the difference between the average response spectrum of the selected accelerograms and the target response spectrum is smaller than a predefined tolerance in a specified interval of structural periods (based on the fundamental period of the system to be analyzed). In most cases, to satisfy spectrum-compatibility, records need to be linearly scaled to a predefined value, which can be the PGA or another selected ordinate of the target spectrum. It is important to emphasize however that the selected records also need to satisfy the requirement of “seismo-compatibility” which means that they must be consistent with the regional seismotectonic and seismogenic setting, as discussed for example in [19].

The rapidly increasing number of good quality strong-motion records seems to make the use of real records a natural and easier choice for practitioners. Moreover, in recent years, several international strong-motion accelerometric databases have been developed, most of which are available over the web, which allow to interactively search events and retrieve waveforms in digital form with prescribed characteristics. Searches can be generally performed using parameters such as magnitude, epicenter distance (or some other definition of distance from the source), site classification, rupture mechanism, peak ground acceleration (PGA), peak ground velocity (PGV), and peak ground displacement (PGD). Specific tools have been developed for the selection of spectrum-compatible suites of real accelerograms, both limited to research purpose (e.g. ASCONA [19]) or available to the general public (e.g. REXEL [20]). Both ASCONA and REXEL-DISP [21] allow for imposing spectrum compatibility either to the acceleration or the displacement response spectrum, being the second option preferable in case of nonlinear analysis.

Despite this, the selection of the appropriate input for time history analysis still requires some skills that are not common for practitioners. For this reason, Rota et al. [22] proposed a web application named SEISM-HOME (SElection of Input Strong-Motion for HOmogeneous MEsozones), available at the internet site www.eucentre.it/seismhome.html, which allows an automatic and prompt definition, at any location of the Italian territory, of the seismic input represented by suites of real spectrum- and seismo-compatible accelerograms recorded at outcropping rock sites with flat topographic surface. However, these records are currently available for the 475 years return period only.

4 IDENTIFICATION OF SUITABLE LIMIT STATES FROM NONLINEAR DYNAMIC ANALYSES OF MASONRY STRUCTURES

The need of identifying suitable performance limit states was early recognized and a rather vague and qualitative definition of the desired performance was described by socio-economic terms. Expressions like “collapse”, “near collapse”, “collapse prevention”, “life safety”, "operational", "fully operational", "immediate occupancy", "damage control" and "serviceability" are used to describe the performance limit states in various documents (e.g. [23]). Nevertheless, as already realized by many authors, these definitions are not appropriate for direct application in numerical analyses and the quantitative translation of the limit states is not straightforward,. For example, Tomaževič [24] observed that it is not possible to quantify the limit states on the basis of the observed damage or of damage indices, which measure the damage occurring to the walls and/or buildings as a whole. Therefore, he tried to establish a correlation between this qualitative definition and the results of experimental studies. Along the same ways, several examples of experimental studies can be found in the literature, where cyclic in-plane tests were performed on masonry piers aiming to describe the different deformation limits at the structural element level for the two damage modes (flexural/rocking and shear failure mode) of the in-plane response of masonry structures (e.g. [25],[26],[27],[16]).

To interpret the results of numerical analyses a quantitative definition of performance levels is necessary, consisting of a proper damage indicator, able to represent the global seismic performance, and adequate damage thresholds expressed in terms of the selected damage indicator.

A quantitative measure of structural performance can be realized with the use of drift/deformation quantities. Such drift thresholds are influenced by the masonry typology, the level of axial loading, the effective boundary conditions and other construction details (e.g. [28]). As discussed for example in [29], displacements and deformations are better indicators of damage than forces and therefore the identification of structural performance levels should be better based on these quantities. In addition to the previous, it is necessary to find significant thresholds of each limit state, that should be expressed in terms of the aforementioned drift quantities and derived from some other measures of structural performance extracted from the results of nonlinear dynamic analyses. Examples of the latter could be some parameter expressing the extension of damage within the different structural elements, or the degradation of the structural response (i.e. in terms of stiffness, lateral strength, etc…) due to progressive damage.

Since a quantitative definition of limit states based on the results of nonlinear static analysis has been proposed by many authors, once the indicators and the thresholds are defined, the results in terms of the selected limit state indicators could be compared to limit state thresholds defined from results of nonlinear static analyses. Examples of some of the adopted definitions of limit states are based on the following quantitative parameters:

1. Significant displacements from the global pushover curve (i.e. the base shear-top displacement curve), as proposed for example in [17]. Specifically, in that work, LS2 was defined as the global displacement corresponding to the attainment of the maximum base shear and LS3 as the displacement corresponding to a shear strength degradation up to 80% of its maximum value (as also suggested in several building codes).
2. Global displacement thresholds corresponding to the attainment of inter-story drift limits. For example, Calvi [30] defined LS2 as the displacement corresponding to the attainment of a maximum inter-story drift of 0.3% and LS3 as the displacement corresponding to the attainment of an inter-story drift of 0.5%.
3. Some indicator of the diffusion of damage, identified for example in [28] by monitoring the level of damage reached in each wall panel.

For the assessment of masonry structures the limit states of interest can be described as:

- LS1-immediate occupancy,
- LS2-damage limitation,

- LS3-life safety,
- LS4-near collapse.

An analytical study with the objective of proposing a suitable definition of significant limit states for masonry buildings, applicable to the results of incremental dynamic analyses (IDA, [31]) was conducted recently by the authors [32].

The study concentrated on the investigation of possible limit state definitions corresponding to the two intermediate limit states (LS2 and LS3) as their identification appears more uncertain and somehow more critical than that of the first and last limits. The identification of LS4 was not addressed, because the definition of the near collapse limit state from the results of numerical analyses is really a difficult task. With reference to nonlinear dynamic analyses, this limit state could be identified by monitoring the IDA curve for each earthquake record and identifying the point for which the slope of the curve approaches zero, as suggested by Ibarra and Krawinkler [33]. However this definition is really vague and it does not easily allow a univocal identification of this limit state. Moreover, as also discussed by Zareian and Krawinkler [34], evaluation of near collapse structural response parameters is strongly related to issues such as assumptions in the structural model, computer program used for the analysis, numerical convergence and stability of the solution. Therefore, the evaluation of LS4 was left aside.

The identification of limit state indicators was approached by applying some proposed criteria to five building models of existing stone masonry buildings. The methodology included the identification of performance limit states from both the results of nonlinear static and dynamic analysis and the comparison of the results obtained in the two cases.

4.1 Drift quantities selected to describe and compare performance levels

To be able to compare alternative definitions of the significant thresholds of structural performance, the significant thresholds derived from different damage quantities need to be expressed by the same drift/displacement quantities. For this reason, in the analytical work of Mouyiannou et al. [32] all the analysis results were interpreted according to two drift quantities, namely the maximum inter-storey drift δ_{max} and a weighted average drift $\delta_w \boldsymbol{\delta}_{\mathbf{w}}$, which were derived from nodal displacements and from element drifts respectively.

The maximum inter-storey drift δ_{max} is the maximum value of pier drift δ_i, derived by the absolute difference of nodal displacements divided by the inter-story height as:

$$\delta_{max} = \max\{\delta_i\} = \frac{\max\left|d_{N_j} - d_{N_i}\right|}{h} \tag{1}$$

where: d_{Nj} and d_{Ni} are the displacements of nodes N_j (top node of the pier) and N_i (bottom node of the pier), respectively, and h is the inter-story height.

The weighted average drift δ_w is calculated as the average of the drifts of all the elements of the critical story, weighted on their area, according to the formula:

$$\bar{\delta}_{\mathrm{w}} = \frac{\sum_{i=1}^{n} (\mathbf{A}_i \delta_i)}{\sum_{i=1}^{n} \mathbf{A}_i} \tag{2}$$

where A_i is the area of the pier i, δ_i is the drift of the pier i and n is the total number of piers of the critical storey, identified as the storey where damage concentrates. The values of δ_i are the element shear drifts, which only account for the shear element deformation, i.e. they are computed by removing the flexural deformation and rigid rotation components from the element drift. This shear drift is an output of the macro-element model, to which the shear behavior with stiffness degradation and strength deterioration is directly related and hence it is considered a suitable indicator of the level of damage in the element. The criteria proposed in [32] for the identification of each limit state are described in the following sections.

4.2 Identification of LS1 (one criterion)

The first limit state can be identified as the displacement that corresponds to the first pier reaching its maximum shear resistance.

4.3 Identification of LS2 and LS3 (3 criteria)

The application of three different criteria was investigated in order to identify limit states which are not directly based on drift quantities, but also described by means of some parameter, representative of the evolution of the structural condition during the nonlinear dynamic analysis. Therefore, each criterion is based on consideration of different damage indicators, all of them trying to synthesize the overall structural behavior, such as for example the extension of damage to the structural elements (criterion 2) or the degradation of the structural response with progressive damage (criterion 1).

As explained in the previous section, the drift quantities corresponding to the attainment of the limit states according to the different criteria were evaluated and compared among each other, to verify whether these definitions of the limit states provide stable and reasonable results in terms of the deformation conditions reached by the structure during the dynamic response. The drifts were also compared to the results of similar criteria applied to pushover analysis.

Regarding the damage limitation state, an extra limitation to the maximum inter-story drift, which should not exceed the value of 0.2% was also adopted.

The criteria proposed to define the damage limitation limit state (LS2) and life safety limit state (LS3) from the results of incremental dynamic analyses (IDA) are discussed in the following.

Criterion 1: identification of LS2 and LS3 from total base shear

The methodology considered for criterion 1 is similar to the definition of limit states from the pushover curve reported in [17], where LS2 and LS3 were identified as corresponding to the attainment of the maximum lateral strength and to its degradation to 80% of the maximum value, respectively. Similarly, for the case of time-history analysis, for each earthquake record analyzed, LS2 is attained at the analysis step for which the shear resistance is reaching its maximum value and LS3 at the step where it drops to 80% of its maximum value. Then the drifts corresponding to the two limit states are derived and their average values among the earthquake records used for the analyses can be calculated. This criterion is very fast and easy to apply and it does not require any engineering judgment or subjectivity, as the definition of the corresponding limit states is quantitative and objective.

Criterion 2: identification of LS2 and LS3 based on the percentage of pier area failing

According to criterion 2, LS2 and LS3 are identified based on the number and percentage of piers achieving the maximum shear drift (predefined value). A reasonable value for maximum shear drift could be 0.4%, as suggested in the EC8-3 [35], although it makes reference to a different definition of element drift. The drift mentioned in the codes is the total element drift, including both shear and flexural components and eventually excluding rigid motions, whilst in the described analytical work the flexural component is removed. The use of the limit of 0.4% indicated in the codes was considered appropriate, because it is derived from experimental evidence of in-plane cyclic tests on mainly squat masonry panels, in which it can be assumed that, when shear failure occurs, the shear deformation component is the one representative of the degradation of the structural response and it is prevailing over the flexural one.

LS2 (damage limitation) is assumed to occur when the first pier reaches the predefined value of shear drift. In order to provide results meaningful for comparison to the results from other criteria, the results are expressed in terms of the average drift values (as defined in section 4.1) calculated from all the earthquake motions used for the analysis.

LS3 is instead corresponding to an appropriately defined level of damage extension, which is expressed in terms of the percentage of the area of the piers that have attained the maximum shear drift with respect to the total pier area, i.e.:

$$A_{fp}[\%] = \frac{\sum_i^m A_{pi}}{\sum_j^n A_{pj}} \cdot 100 \qquad (3)$$

where m is the number of the piers attained the maximum shear drift and n is the total number of piers. For each PGA level considered for the analyses, and for each earthquake record, the percentage of the pier area failing is calculated and compared with a predefined target percentage. For each considered building, LS3 is then attained when the average (among all the earthquake records used) percentage area reaches the predefined drift limit.

Attention should be taken when evaluating the appropriate target percentage area for LS3 since the procedure is strongly dependent on its definition which needs to be identified case by case and whose value cannot be considered as general. The value needs to be selected based on engineering judgment and should be associated with drift values which are in accordance with those derived from nonlinear static analyses. In addition, this percentage should guarantee collapse prevention, i.e. limited lateral strength degradation, since the criterion is applied to identify the life safety condition. A reasonable percentage, representative of the results, was considered to be 50% of the total pier area in the direction of analysis.

Criterion 3: identification of LS2 and LS3 from PGA-drift curves

The third criterion is applied to the so-called IDA curves which form an alternative representation of the results of incremental dynamic analyses, reporting the level of PGA versus an appropriately defined drift quantity. Each curve is a multi-linear curve obtained by joining the drift values calculated for subsequent levels of PGA examined. The average PGA-drift curve for the critical story of each building can be evaluated by plotting the average drifts among the earthquake records for each PGA.

LS2 can be identified at the first significant change of slope in the average curve. This change of slope is related to an increased rate of drift variation as a function of PGA, which can be seen as representative of an increase of structural damage.

LS3 can be identified as the range of drifts between which the slope of the curve degrades reaching a predefined percentage of the initial slope. This percentage is selected according to engineering judgment in order to represent a damage level adequate for the life safety limit state and should provide drift values in agreement with the results of nonlinear static analysis. Based on the results obtained in the considered analytical study and specifically on the comparison of the corresponding drift values with those derived from the results of nonlinear dynamic analyses by applying the other criteria and the results of nonlinear static analyses, a percentage equal to 7% of the initial slope was selected (after having tried different values up to 10%).

4.4 Application of limit state identification criteria to the results of nonlinear dynamic analysis of existing masonry buildings

The identification criteria for the three limit states discussed above were applied to 5 building prototypes, which are briefly presented in the following. The main outcomes obtained from the application of the criteria are then discussed.

4.4.1 Building prototypes, modeling assumptions and selected ground motions

Five building configurations were selected as representative of different structural typologies of unreinforced stone masonry structures. Their models are shown in Figure 4. All the analyses were performed is the x-direction indicated in the figure.

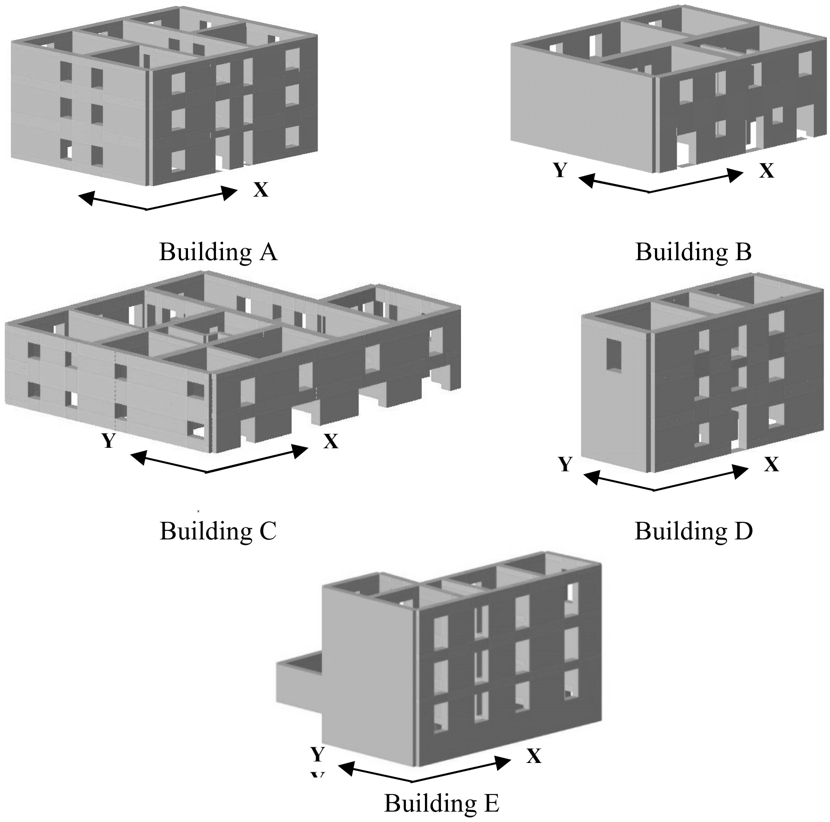


Figure 4 Analyzed structural configurations (after [32]).

All buildings were assumed to have stiff diaphragms, an assumption consistent with the adopted modeling approach which only considers a global type of response, which is not governed by the out-of-plane behavior of walls. Out-of-plane failure mechanisms are assumed to be prevented by proper connections and detailing.

The mechanical properties of stone masonry adopted in the model were defined according to the specimens of an extended experimental campaign carried out in Pavia in the last years ([36][37]), consisting of a wide characterization campaign (including tests on mortar, vertical compression and diagonal compression tests on wallettes, cyclic shear compression tests on walls), followed by full-scale shaking table tests on three prototype buildings [38][39]. The average experimental values of the elastic modulus, E, the shear modulus, G, the masonry density, ρ, and the compressive strength of masonry, f_m have been used. The values used for the initial shear resistance for zero compression, f_{v0} and the friction coefficient, μ were instead obtained from the calibration of the macro-element model on the results of cyclic in-plane tests of masonry piers [26].

Incremental dynamic analyses were performed using seven real earthquake records, scaled to increasing values of PGA (from 0.05g to 0.60g) to represent different levels of seismic severity. The selection of the real spectrum-compatible accelerograms was made using the algorithm of the ASCONA program [19]. The real records were selected to be compatible in the mean with the EC8-1 [1] type 1 acceleration response spectrum. The spectrum was anchored to a PGA of 0.2g, selected to be approximately a central value of the seismic intensities considered for the analyses. This choice was based on the attempt of limiting the scale factors applied to the records, as these seven records were then scaled to the different levels of PGA for which nonlinear dynamic analyses were carried out.

4.4.2 Resulting drift thresholds for LS1, LS2 and LS3

LS1 was identified as the state corresponding to the first pier reaching its maximum shear strength as reported in section 4.2. The average values (between the 7 earthquake records analyzed) of maximum element drifts corresponding to LS1, are presented in Figure 5.

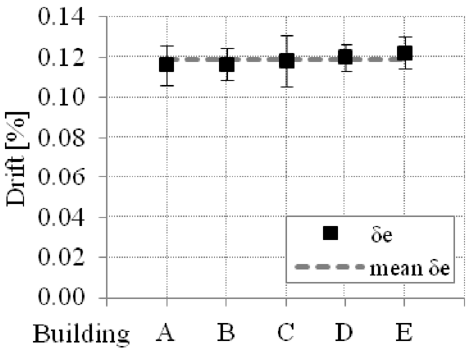
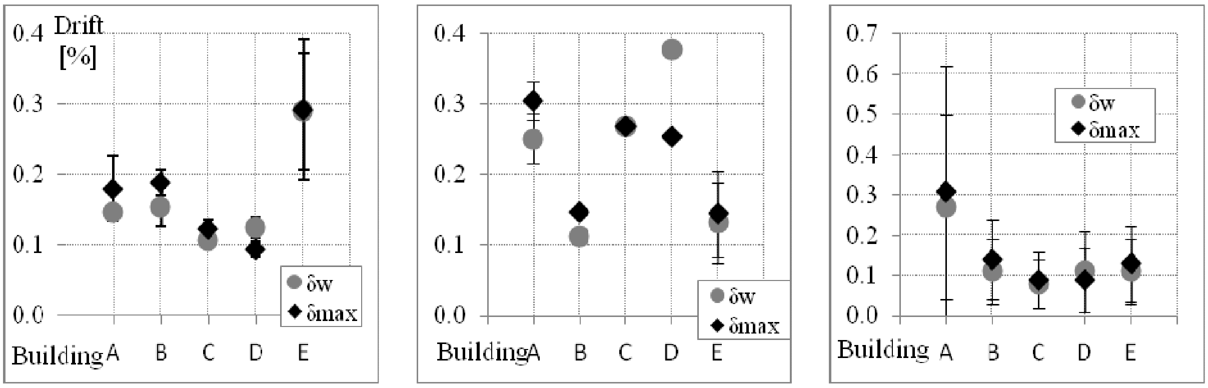


Figure 5 Maximum element drifts corresponding to the attainment of LS1 (average of 7 earthquake records)

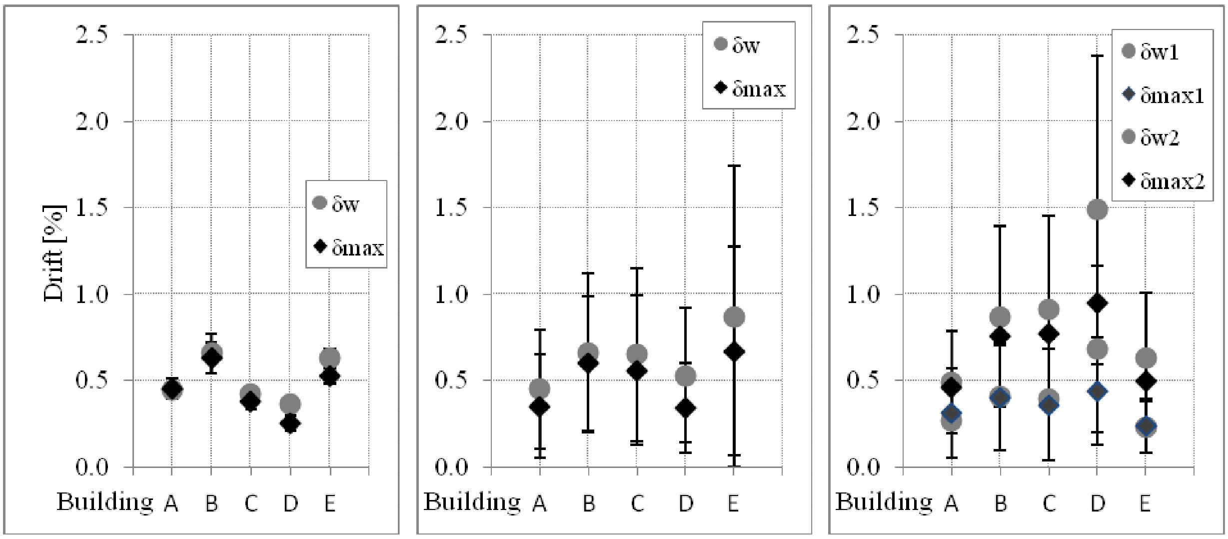
The average value derived from all buildings is 0.12%, in agreement with the experimental results obtained in [27], according to which the maximum shear resistance of a stone masonry element is reached for a maximum element drift in the range of 0.10-0.15%. The results confirm that the drifts corresponding to LS1 are not depending on the building typology or on the earthquake records, as they are only a property of the numerical model and of the masonry typology.

The drift values corresponding to the attainment of the damage limitation limit state and life safety limit state were identified by applying the three identification criteria as described previously. The derived drift quantities, namely the average (between the earthquake records used) maximum inter-story drift (δ_{max}) and weighted average story drift (δ_w), resulted by the application of the criteria for the identification of LS2 and LS3 are presented in Figure 6 and Figure 7 respectively for all the buildings analyzed. The drift values noted with black diamonds correspond to δ_{max} and grey circles to δ_w. The error bars represent the coefficient of variation (C.o.V) resulting from record-to-record variability.



a) Criterion 1 b) Criterion 2 c) Criterion 3

Figure 6 Average drift quantities resulting from the application of criteria for the identification of LS2



a) Criterion 1 b) Criterion 2 c) Criterion 3

Figure 7 Average drift quantities resulting from the application of criteria for LS3

As observed in Figure 6.a), the application of criterion 1 for LS2 identification, results in values of both the drift quantities between 0.1 and 0.2%, with the only exception of the drifts for building E. In this case, both drifts are equal to 0.29%, which in case of δ_{max} exceeds the limit value of 0.2%. However the limits for building E are characterized by the largest coefficient of variation. Apart from the case of building B, the C.o.V. of the values of δ_w is always lower than that of δ_{max}, although their values vary significantly from building to building.

The range of drift values obtained by the application of criterion 2 is larger than the range of drift values of LS2 resulting from the application of criterion 1, with values between 0.1 and 0.38%, as shown in Figure 6b. Nevertheless the values of C.o.V are lower than the values resulting from the other criteria, indicating a small dependence of the application of criterion 2 to the record-to-record variability.

Regarding the average drift values obtained by the application of criterion 3 for LS2 (Figure 6.c), similar values around 0.1% are observed for all buildings except for building A, which attained a significantly higher value. It has to be underlined that the C.o.V has very large values for the application of criterion 3, showing a high dependency of the results of the criterion on the record-to-record variability.

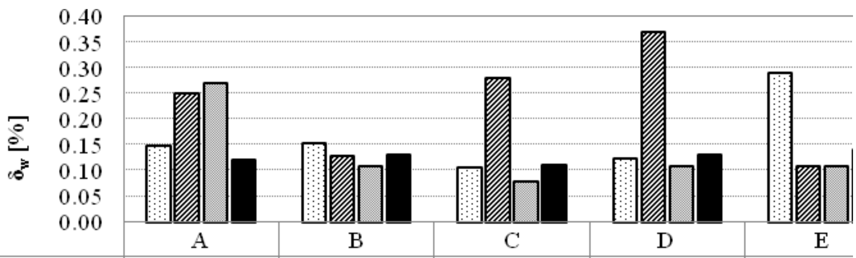
The drift values corresponding to LS3, derived by the application of criterion 1 are ranging between 0.25% and 0.65%. The maximum C.o.V.is found for the case of δ_w for building B, and it is equal to 17.5%, which is small compared to the values of C.o.V resulting from the application of the other criteria for the derivation of LS3. It can be noted that, as observed from the application of criterion 1, the average values of the two drift quantities for LS3 are quite similar to each other, with the exception of building D.

The drift values of LS3 obtained by the application of criterion 2 (shown in Figure 7.b.) correspond to the level of PGA for which the average percentage (among the results from seven earthquakes) of piers reaching the maximum shear drift exceeds the 50% of the total pier area. From the values obtained, a rather wide range of drift is noticed, with buildings A and D having similar drift values at the lower bound of the range and buildings B, C and E having similar values at the higher bound of the range. It is important to notice the very large C.o.V. resulting for all buildings, indicating the significant dependence of the results of this criterion on the record-to-record variability.

As previously explained the results of criterion 3 are expressed by two values of drift corresponding to the upper and lower value of the drift range for which the slope of the IDA curve drops below a predetermined percentage of the initial slope. The results (reported in Figure 7.c) show a significant variability from building to building both in terms of the range width and of the values corresponding to the upper and lower bound drift values. This is related to the level of discretization of the PGA values used for the analyses and to the slope of the curve in the drift range of interest. Also, the very large values of C.o.V. indicate the strong dependency of the results on the record-to-record variability.

4.5 Comparison of the applied criteria and selection of the optimal criteria for LS2 and LS3 identification

A comparison between the results derived by the application of different criteria to the results of nonlinear dynamic analysis and also to the results of nonlinear static analysis is visualized with the histograms from Figure 8 to Figure 11. The first two histograms correspond to the results for LS2 and represent the values of weighted average story drift and its C.o.V due to the record-to-record variability. The choice of reporting only the results for LS2 in terms of weighted average story drift is based on the fact that it provided a better match with the results of pushover analyses than the maximum inter-story drift.



	A	B	C	D	E
Crit. 1 [TH]	0.15	0.15	0.11	0.13	0.29
Crit. 2 [TH]	0.25	0.13	0.28	0.37	0.11
Crit. 3 [TH]	0.27	0.11	0.08	0.11	0.11
Cyclic PO	0.12	0.13	0.11	0.13	0.14

Figure 8 Weighted average drift limits for LS2 derived from the results of nonlinear dynamic analysis by applying the three identification criteria and from the results of nonlinear static analysis.

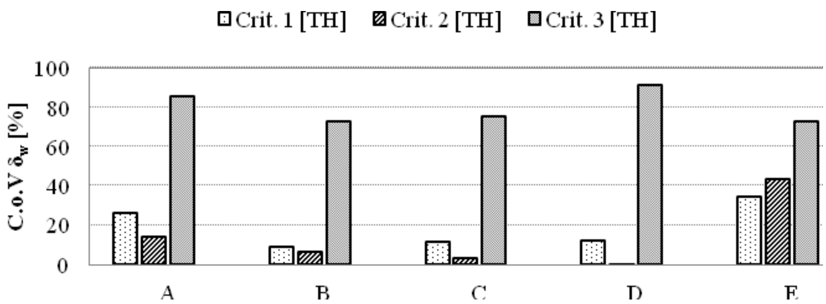
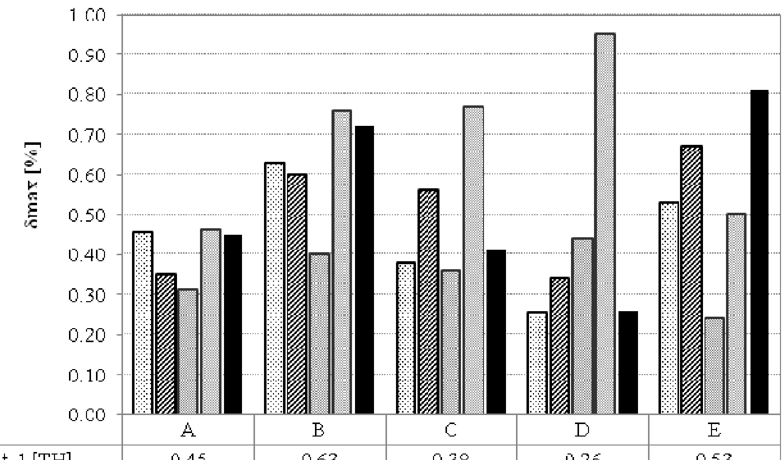


Figure 9 Coefficient of variation of the values of weighted average story drift limits of LS2, due to record-to-record variability.

Looking at the results obtained for LS2 in terms of weighted average drift as presented in Figure 8, it can be noted that criterion 1 and criterion 3 are reproducing quite well the results obtained from pushover analysis, with the exception of building E and A, respectively for the two criteria. Criterion 2 provides higher values of drift than all other criteria for buildings C and D and, in general, a good agreement with the results obtained from pushover analysis cannot be found.

As can be observed by Figure 9 criterion 3 has a significantly higher value of C.o.V. with respect to the others. For both criteria 1 and 2, building E has significantly higher values of C.o.V., which may be related to the marked structural irregularity of the building. Consequently both criterion 1 and criterion 2 appear to be suitable for the identification of LS2, always combined with the limitation of the maximum inter-story drift to the value of 0.2%.

A comparison of the drift limits provided by the different criteria for LS3 is shown in Figure 10 and Figure 11 in terms of the average value of maximum inter-story drift and its C.o.V due to record-to-record variability respectively. For this limit state, results are presentedin terms of maximum inter-story drift, since this drift shows a better agreement with the results of nonlinear static analysis.



	A	B	C	D	E
Crit 1 [TH]	0.45	0.63	0.38	0.26	0.53
Crit 2 [TH]	0.35	0.60	0.56	0.34	0.67
Crit. 3-min[TH]	0.31	0.40	0.36	0.44	0.24
Crit. 3-max[TH]	0.46	0.76	0.77	0.95	0.50
Cyclic PO	0.45	0.72	0.41	0.26	0.81

Figure 10 Maximum inter-story drift values for LS3 derived from the results of nonlinear dynamic analysis by applying the three identification criteria and from the results of nonlinear static analysis.

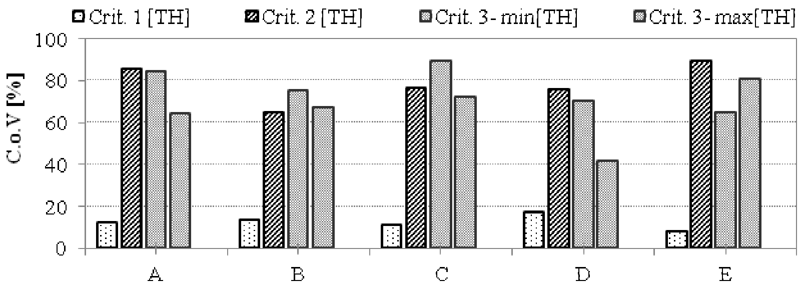


Figure 11 Coefficient of variation of the values of maximum inter-story drift for LS3, due to record-to-record variability.

The histogram of Figure 10 shows that the different criteria provide different results in terms of attained drifts, as expected since each criterion is based on consideration of different quantities. In order to apply criteria 2 and 3, some reasonable target values has been assumed for the percentage of pier area failing and the percentage of the initial slope, respectively. It is generally noticed that there is no unique value of percentage to be set for criteria 2 and 3 to guarantee that the requirements of the other criteria will be met for all buildings. For the application of criterion 3 to identify LS3, the considered target value of the slope reduction with respect to the initial slope (7%) leads to drift ranges which are not always in agreement with the results of criterion 1 and 2. In case of the maximum inter-story drift, even for building E the value produced by criterion 1 is outside the range of criterion 3. Criterion 2 provides instead results within the range defined by criterion 3 only for buildings A, B and C.

The results obtained from pushover analyses are in general more consistent with the results from criterion 1 rather than with those obtained from the other two criteria, with the only exception of building E. This similarity of the results could be expected as criterion 1 is analogous to the definition of limit states used for pushover analyses. Regarding building E it should be noted that the structure is irregular in plan and in elevation and therefore the application of nonlinear static analysis is questionable and the validity of its results is not guaranteed.

Finally by observing Figure 11 it is obvious that the drifts derived from criterion 1 for LS3 have a coefficient of variation much smaller than that corresponding to the other criteria.

The aforementioned difficulties in the application of criteria 2 and 3, in addition to the significantly larger coefficient of variation of the drifts obtained by these criteria, led to the selection of criterion 1 as the optimum for the identification of LS2 and LS3 from the results of nonlinear dynamic analysis. Criterion 1 is indeed the criterion providing the most stable and consistent results and it is the least dependent on the record-to-record variability. Moreover, it is equivalent to the definition of LS2 and LS3 based on the results of nonlinear static analyses and it is the most straightforward to apply, as it does not require any particular engineering judgment or the definition of target values. (i.e. the percentage of pier area failing for criterion 2 and the percentage of slope reduction for the PGA-drift curves for criterion 3).

5 CONCLUDING REMARKS

As stated in the introduction, the use of nonlinear time-history analysis for masonry structures requires suitable modeling approaches and an appropriate selection of input ground-motion records. The latter is an issue common to all structural types and several solutions are available.

The TREMURI computer program, concisely presented in section 2, includes modeling and analysis features specifically developed for the dynamic analysis of entire masonry buildings.

What is still missing is a well-defined method for identifying the structural performance levels based on damage and/or displacement/deformation indicators, which would help in the interpretation of the results of dynamic analyses and may support a broader application of the performance-based procedure with incremental time-history analysis of masonry buildings.

A procedure for the identification of limit states has been presented in this paper. The first limit state considered, i.e. immediate occupancy (LS1), was identified as corresponding to the first pier reaching its maximum shear strength. The definition of LS2 (damage limitation) and LS3 (life safety) from the results of time-history analyses was more problematic and therefore three different criteria were proposed and tested, each one concerning requirements on different quantities. The first criterion was based on global lateral strength evolution, the second criterion on damage diffusion and the third criterion on the degradation of the structural response for increasing levels of ground motion. Each of these parameters was considered to be a good descriptor of the global structural performance, taking into account the overall behavior of the considered buildings. In order to compare the results of the different criteria (which are based on completely different quantities) and to make sure that they provide reasonable results in terms of deformation capacity, conveniently defined drift quantities were associated with the limit states identified with the different criteria

For each limit state, the best criterion was selected together with the associated drift quantity providing the most stable (and the least dependent on the record-to-record variability) and consistent results, i.e.:

- LS1 identified as corresponding to the first pier attaining its maximum shear resistance;
- LS2 defined as the average weighted story drift corresponding to the attainment of the maximum base shear;
- LS3 identified as the maximum inter-story drift corresponding to a 20% degradation from the maximum value of base shear.

The reported study was limited to a small number of building configurations and a specific masonry typology. Other important parameters should also be explored in order to verify the adequacy of the proposed criteria for the identification of relevant limit states. In addition to the previous, the consistency of the proposed approach with experimental results and empirical observations should be verified.

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