Seismic assessment of plan irregular masonry buildings with flexible diaphragms

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ABSTRACT: The seismic Performance-Based assessment of existing masonry buildings requires the use of nonlinear models, in order to check the attainment of ultimate limit states. Incremental Dynamic Analysis represents the most accurate method but very few models are available which are able to describe the stiffness and strength degradation, which are typical of masonry buildings, as well as the hysteretic behaviour of piers and spandrels under cyclic actions. At engineering practice level, the Displacement-Based approach is widely adopted, through the use of nonlinear static analysis. However, the application in the case of irregular URM buildings with flexible horizontal diaphragms represents an open issue, due to various difficulties, for example in the transformation of the pushover curve of the original MDOF into the equivalent SDOF or in the definition of performance levels. A wide numerical investigation was made on some case studies, in order to check the applicability of nonlinear static analyses and propose some new procedures. Nonlinear dynamic analyses have been adopted as reference solution.

1 INTRODUCTION

The Performance-Based Assessment (PBA) requires the evaluation of the seismic response to earthquake of different intensity, till to near collapse conditions. Since masonry has a strongly nonlinear behaviour, also for low horizontal actions, the use of a Displacement-Based Approach (DBA) turns out to be necessary for a reliable assessment, which requires the availability of numerical models for static pushover or dynamic nonlinear analyses. In addition, proper criteria for the definition of Performance Levels (PLs) are necessary, which are not straightforward in complex masonry buildings, often irregular in plan and/or in elevation, as well as characterized by the presence of flexible horizontal diaphragms. Nonlinear static analysis, that is widely adopted in international standards (e.g. ASCE/SEI 41-13 2014, EC8-1 2004, NTC 2008), has been originally developed for RC or steel framed structures, under the hypothesis of rigid horizontal diaphragms and, possibly, in the case of regular configurations. Therefore, in case of buildings which present plan and/or elevation irregularities and with flexible diaphragms, questions arise on the reliability of this procedure. In this paper the focus is pointed at three steps of this methodology: (i) the effect of the transformation of the pushover curve of the original MDOF to the equivalent SDOF; (ii) the definition of the performance levels; (iii) the definition of the seismic demand. About the issues (i) and (iii) the procedure foreseen by the European codes (Eurocode 8 and NTC 2008), based on inelastic spectra, was compared with the procedure that uses overdamped spectra (similar to the one adopted in ASCE/SEI 41-13 2014). About the issue (ii) the results derived by the European codes and recent proposals available in literature (the *multiscale approach* proposed in Lagomarsino and Cattari 2015a) were compared.

The results achieved in a wide parametric analysis on different prototypes of masonry buildings are presented. These models were conceived starting from a regular configuration, then a progressing increase in the plan irregularity and a decrease in the stiffness of diaphragms were applied. Then the effect consequent to add ring beams was also studied. For each case study, the results obtained by nonlinear Incremental Dynamic Analyses (IDA) are considered as the reference correct behaviour.

2 CASE STUDIES AND APPROACH ADOPTED FOR THE ASSESSMENT OF THE RELIABILITY OF NON LINEAR STATIC PROCEDURES

Non linear static analyses on 10 numerical models have been carried out with the aim to verify the reliability of this type of procedure. The result of non linear dynamic analysis was used as reference solution. The models represent a 3-storeys URM building and were modeled following the equivalent frame approach (Lagomarsino et. al 2013). Starting from a base model, with a regular plan configuration with rigid diaphragms (representative of RC slabs) and chains at each level, variations were subsequently added. Those variations are: (i) the substitution of the rigid diaphragms with more flexible solutions representative of wooden or vault diaphragms; (ii) the introduction of a plan irregularity changing the strength and the stiffness of two outer walls; (iii) the substitution of the chains with ring beams at each level. Combining all these variations 10 models were developed from the base configuration. In figure 1 the plans of the regular and the irregular configurations with the 3D view of the model are depicted. In table 1 a summary of the main mechanical properties of these models is reported, being illustrated a more detailed description in Cattari et al. 2015.

Table 1. Summary of the significant mechanical properties of the models

Masonry properties		**Diaphragms properties**			
Young modulus	$E_m = 750$ MPa	$E_1 = 58800$ MPa	Young modulus in the joist direction		
Shear modulus	$G_m = 250$ MPa	$E_2 = 30000$ MPa	Young modulus in the direction perpendicular to E_1		
Compressive strength	$f_m = 2.80$ MPa	$\nu = 0.2$ [-]	Poisson ratio		
Equivalent cohesion*	$\tilde{c} = 0.11$ MPa	t = 4 cm	Thickness of the equivalent orthotropic membrane		
Equivalent friction*	$\hat{\mu} = 0.34$ [-]		Rigid	Intermediate	Flexible
Mass density	$\rho = 18$ kN/m^3	Shear modulus	$G_m = 12500$ MPa	$G_m = 100$ MPa	$G_m = 10$ MPa

* The values of the cohesion $\tilde{c}$ and the friction $\hat{\mu}$ are obtained from Mann and Muller proposal (1980)

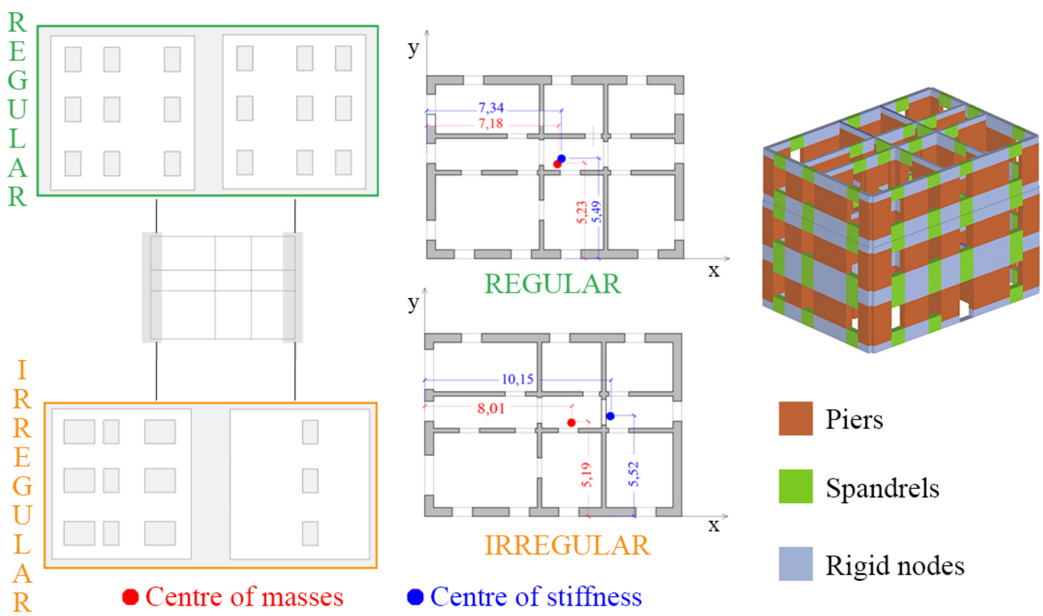

Figure 1. Differences between regular and irregular model. Plans and 3D view

The pushover analyses have been carried out in each model with five load patterns kept invariant during the analysis. In particular, three of the load patterns used were proportional to: the masses (*uniform*); the product mass x height (*pseudo-triangular)*; the fundamental modal shape. Then two additional load patterns have been investigated, obtained by a combination of load patterns derived from all mode shapes which do not present the inversion of sign in displacement (modes of 1st type): 1) SRSS, using the Square Root of Sum of Squares of the 1st type modes; 2) CQC, using a Complete Quadratic Combination of the same modes. The steps forward foreseen by the non linear static procedure are: (i) to transform the nonlinear force-displacement relationship between the base shear and the displacement of the control node into an equivalent relationship representative of an idealized SDOF (see section 2.1), (ii) a proper identification of the damage levels (DLs) and related performance levels (PLs), discussed in section 2.2.

In the case of nonlinear dynamic analysis the seismic input is the acceleration time-history at the base of the structure. Ten records have been used, compatible with the accelerations expected in L'Aquila (Italy). Those are conditioned to the spectral acceleration S_a for the period T=0.36 s, assumed as representative of the main modes of vibration of the considered buildings. Incremental dynamic analyses (IDA) have been performed (Vamvatsikos and Cornell 2002), obtaining for each record the value of the intensity measure corresponding to the attainment of each performance level (IM_{PL}), being this latter calculated with the multiscale approach (see section 2.2). Finally the median value and 16% and 84% percentiles of IM_{PL} for the ten records are evaluated, under the hypothesis IM_{PL} is log-normally distributed. IM_{PL} can be evaluated as the intensity measure (IM) for which the spectral displacement demand S_d is equal to the displacement "d" that causes the attainment of a certain DL ("d" is the displacement of the capacity curve, that is the original displacement "u" of the pushover curve properly converted in the SDOF system).

2.1 Criteria used to transform the initial MDOF to the equivalent SDOF

Italian (NTC 2008 & Circ 617/2009) and European (EN 1998-1 2004 & EN 1998-3 2005) codes define a similar methodology in order to carry out the seismic assessment of an existing URM building using the non linear static procedure. It is derived from the so called "N2 method" developed by Fajfar in the '90s (Fajfar 2000).

After the pushover analysis has been carried out, the first step of the procedure is to transform the nonlinear force-displacement relationship between the base shear and the displacement of the control node in an equivalent elasto-perfectly plastic relationship representative of an equivalent SDOF. The mass of the equivalent SDOF (m^*) and the transformation factor (Γ) are determined as:

$$m^* = \Sigma m_i \phi_i \tag{1}$$

$$\Gamma = \frac{m^*}{\Sigma m_i {\phi_i}^2} \tag{2}$$

where m_i is the mass associate to the i-th node of the numerical model and ϕ_i is the correspondent normalized displacement of the fundamental modal shape.

The force F* and the displacement d* of the equivalent SDOF model are computed as:

$$F^* = \frac{F_b}{\Gamma} \tag{3}$$

$$d^* = \frac{d_n}{\Gamma} \tag{4}$$

where F_b and d_n are, respectively, the base shear force and the control node displacement of the MDOF system. Then the period T* of the equivalent SDOF is determined by:

$$T^* = 2\pi \sqrt{\frac{m^*}{k^*}} \tag{5}$$

where k^* is the stiffness of the equivalent SDOF, calculated as the slope of the line segment passing through the origin and the point corresponding to the 60% maximum base shear force at any point along the force-displacement curve (NTC 2008). In the Eurocodes the procedure is similar, the period T^* is calculated with the yield displacement d_y^* and the yield force F_y^*. The value of F_y^* - according to the bilinear idealization of the original pushover curve – is herein computed by imposing the area equivalence.

At this stage it is possible to determine the target displacement from the elastic displacement response spectrum for the equivalent SDOF with period T^*. However, if $T^* < T_C$ it is necessary to take into account the nonlinear response through the use of the coefficient q_u defined as:

$$q_u = \frac{S_e(T^*)m^*}{F_y^*} \quad (6)$$

An alternative approach derives from the so called *Capacity Spectrum Method* (CSM) originally proposed by Freeman in 1978; the seismic demand is an overdamped spectrum and the transformation of the MDOF in a SDOF does not foresee the conversion of the pushover curve in an equivalent bilinear relationship, but only the conversion into the equivalent SDOF that may be carried out with analogous criteria to those expressed by Equs. (3) and (4). Thus, a strong advantage of such procedure is that it is not dependent by the period T^* as in N2 method.

2.2 **Criteria used to define the performance levels**

In order to check the fulfillment of the considered PL, the corresponding DL has to be positioned on the pushover curve, by using all information provided by the incremental nonlinear static analysis. This is a complex task, which is tackled by codes and recommendation documents according to the following main approaches:

- *Structural element approach*. It assumes that the attainment of a certain DL in the building corresponds to the step in which the first structural element reaches the same DL. This approach, used in the American standards as ASCE/SEI 41-13 2014, This is particularly useful when the mechanical model is not able to capture the progressive strength degradation of the pushover curve.
- *Heuristic approach*. DLs are directly defined on the pushover curve on the basis of conventional limits, usually expressed in terms of interstorey drift and decay fraction of the overall base shear. This approach is used in the European standards (EN 1998-3 2005 & NTC 2008) and requires the use of shear-drift constitutive relations with strength degradation and limited ductility.

For seismic assessment purposes, the Italian code follows the heuristic approach and requires the safety checks for two performance levels:

- life safety limit state (LS): the ultimate displacement capacity is taken as the roof displacement at which total lateral resistance (base shear) has dropped below 80% of the peak resistance of the structure;
- damage limitation (DL): the capacity for global assessment is defined from the first of the conditions (i) the yield point (yield force and yield displacement) of the idealized elasto-perfectly plastic force-displacement relationship of the equivalent SDOF system, (ii) the displacement corresponding to the reaching of the threshold equal to 0.3% of the interstorey drift in a wall.

In the Italian code a near collapse limit state is also foreseen, although no specific indications are suggested to define it on the pushover curve. In the European code (EN 1998-3 2005), in case of assessment of existing buildings, also three performance levels are defined, conceptually consistent with those prescribed in the Italian code.

The *heuristic approach*, however, may result quite conventional and not reliable if adopted as single criterion to define the DLs on the pushover curve. This because it does not detect the occurrence of heavy DLs at local or macroelement scale (i.e. each single masonry wall). In particular, if the building is very large and horizontal diaphragms are flexible, a significant damage in one single wall may not appear evident in the pushover curve of the whole structure. The same applies for damage in structural elements, which can spread too much in the building without any tangible effect in the global pushover curve. On the other hand the application of the *Structural element approach* may provide very conservative results, in fact even if one structural element is heavily damage the construction may still be functional.

Therefore, in addition to the criteria proposed by the Italian and European codes, in this paper the multiscale approach developed within the PERPETUATE research project (Lagomarsino and Cattari 2015a) has been adopted, which takes into account the behavior of single elements (E), macroelement (M) and of the global building (G). For each scale, proper variables are introduced and their evolution in nonlinear phase is monitored: the percentage of panels (piers and spandrels as identified in the equivalent frame idealization of URM walls) that reach or exceeds a certain damage level checked thorugh the reaching of given threshold of drift at structural element scale (E); the interstorey drift in masonry walls and the angular strain in horizontal diaphragms (M); the normalized total base shear, from global pushover curve (G). The reaching of assigned thresholds for such variables allows to define the displacements on the pushover curve corresponding to the attainment of PL at these different scales, being thus the minimum value that establishes the final position of PL. The adoption of the multiscale approach turns out very useful in particular when a damage concentration is expected on single walls that however could not correspond to a significant decay of the overall shear base. A summary of the thresholds herein used to verify the reliability of the non linear static methodology is reported in table 2. The multiscale approach may be adopted with consistent criteria in case of both static and dynamic analyses (Lagomarsino and Cattari 2015b).

Table 2. Thresholds used for the definition of the Performance Levels.

	EC 8 – NTC 2008		**Lagomarsino and Cattari 2015**	
Scale of analysis	Damage limitation	Life Safety	Damage limitation	Life Safety
Local	-	-	3%	3%
Macroelement [max drift]	0.3%	-	0.3%	0.5%
Global [base shear]	100 %	80 %	100 %	80%

2.3 Approach used to define the seismic demand

In order to define the elastic seismic demand the median spectrum derived from the 10 records adopted for the non linear dynamic analyses was used. Then, in order to reduce this spectrum to take into account the non linear behavior of the building both the CSM and the N2 approach were used. With the first approach, used in US and New Zealand codes, the elastic spectra is reduced considering that the damping increase with the damage of the building. For this reason, an analytical law that relates, for each step of the pushover, the period T^* and the equivalent damping ξ has been adopted, herein calibrated on basis of cyclic pushover analyses performed on each model.

In the second approach, instead, the elastic spectrum is reduced considering the ductility of the equivalent SDOF, computed on basis of Equ.(6).

2.4 Summary of the procedure adopted

So, as a sum up, non linear static analyses on 10 URM models were carried out. The force-displacement relationship obtained for each model and for each load pattern was transformed in the capacity curve, representative of the equivalent SDOF. Then, the value of IM_{PL} from nonlinear static analyses was calculated with both the CSM and N2 methods and then compared with the IM_{PL} of the non linear dynamic analyses, calculated with the multiscale approach, considered as reference solution.

However, as far as the conversion into SDOF concerns, the Γ and m^* coefficients were calculated differently from the procedure described in the section 2.1. Indeed it is foreseen that these coefficients are usually calculated, for any load pattern, from the first modal shape. This choice, however, seems rather arbitrary, especially in the case of flexible diaphragms, when the first modal shape does not activate all the walls of the building. For this reason, the displacement profile produced in the elastic phase by the application of each load pattern assigned has been adopted as reference for the conversion.

It is worth underlining that it is possible to calculate the IM_{PL} not only in correspondence of the limit states, but it could be calculated for each displacement of the pushover curve. With this procedure the ISA (incremental static analysis) is obtained. By comparing ISA and IDA curves it is possible to provide a more comprehensive comparison of results between static and dynamic nonlinear analyses. In particular, it is useful to highlight if the possible differences in values of IM_{PL} are mainly related to discrepancies in the attainment of PL, for example due to a different spread of damage simulated by two types of analyses, or to those related to intrinsic limits of the static method (e.g. on the conversion into equivalent SDOF, the approximate evaluation of damping, etc.). In fact, in the first case the IDA and ISA curves are expected to be very similar, while not in the second one. Examples of this comparison are in figure 2.

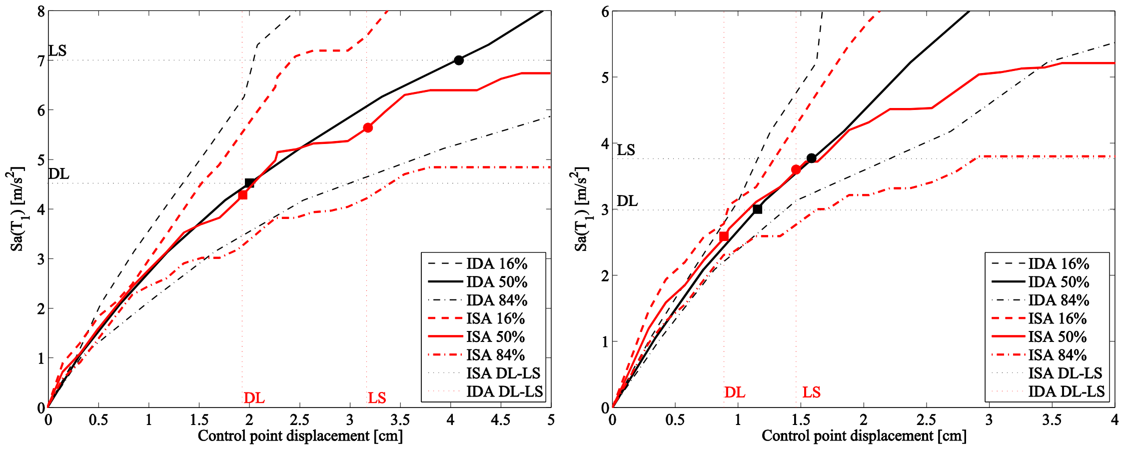

Figure 2. Examples of IDA-ISA curves comparison. Left for the irregular building with intermediate diaphragms and chains at each level, right with ring beams. Both ISA curves derive from the uniform load pattern.

3 SUMMARY OF THE RESULTS

For each model the dynamic result adopted as reference corresponds to the median value of the IM_{PL} from all the records used. From the comparison between the static and the dynamic results it is possible to understand which load pattern shows the closest result to the dynamic one. To this aim, the ratio IM_{st}/IM_{dyn} has been introduced: if this ratio is >1 it means the non linear static analysis provides a value of IM higher than the non linear dynamic analysis. On the contrary, the non linear static analysis gives conservative results if the ratio IM_{st}/IM_{dyn} is <1.

In figure 3 an example of the diagram used to compare the static and dynamic results is shown. In the horizontal axis the 10 models under analysis are listed, each with an acronym: letter "A" refers to the model with chain; letter "B" refers to the model with ring beams; letter "r" refers to the models with a regular plan configuration whereas with "ir" to the irregular plan configuration. In the vertical axis there is the ratio IM_{st}/IM_{dyn}. The part of the diagram where this ratio is bigger than 1 is highlighted in orange because it means that the non linear static analysis provides non conservative results.

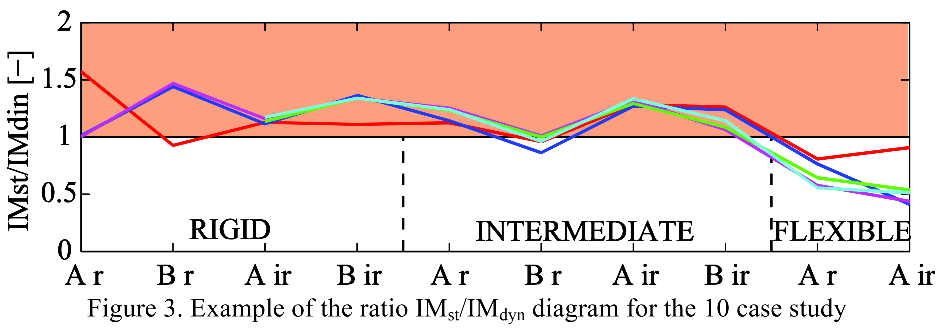

Figure 3. Example of the ratio IM_{st}/IM_{dyn} diagram for the 10 case study

In figure 4 and 5 the diagrams for the DL and the LS limit states are reported. On the left the demand was calculated with the overdamped spectra, on the right with the inelastic ones: in both cases, the criteria proposed in European codes have been adopted. If for the DL limit state there are not strong differences between the two methods, for the LS limit state the N2 Method provides non conservative results. Still with the overdamped spectra, when the diaphragms are flexible, many load patterns provide non conservative results. However, under the hypothesis to consider at least two load distributions and choosing one of two as the uniform one, the use of the non linear static method for the assessment of URM buildings provide reliable results.

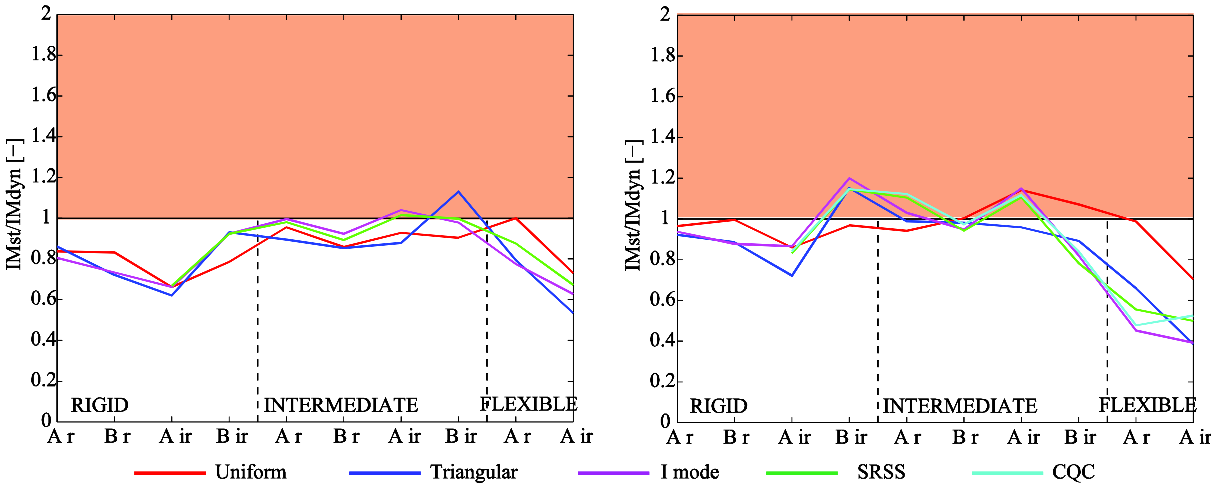

Figure 4. Damage limitation limit state, ratio IM_{st}/IM_{din}. Left calculated with CSM, right with N2 method by adopting the criteria proposed in European codes for defining the PLs

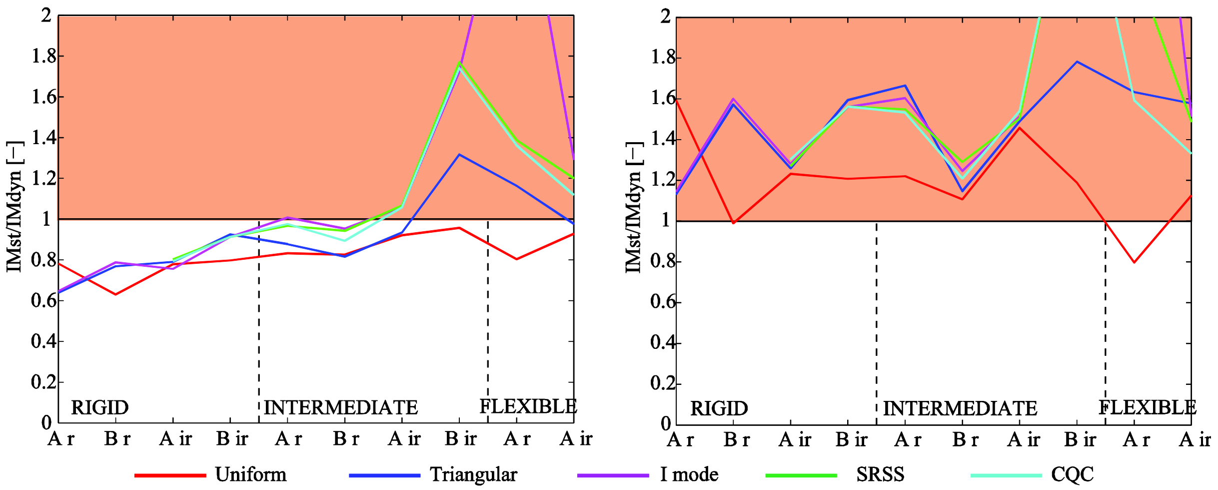

Figure 5. Life safety limit state, ratio IM_{st}/IM_{din}. Left calculated with CSM, right with N2 method by adopting the criteria proposed in European codes for defining the PLs

In figures 6 and 7 always the DL and the LS limit states are reported, but in this case the criteria of the multiscale approach are used. If for the DL limit state there are not so strong differences, for the LS limit state to use the multiscale approach turns out to be crucial. Indeed with this approach to use the inelastic spectra provides more conservative results, although once again the use of overdamped spectra guarantees results that are more reliable.

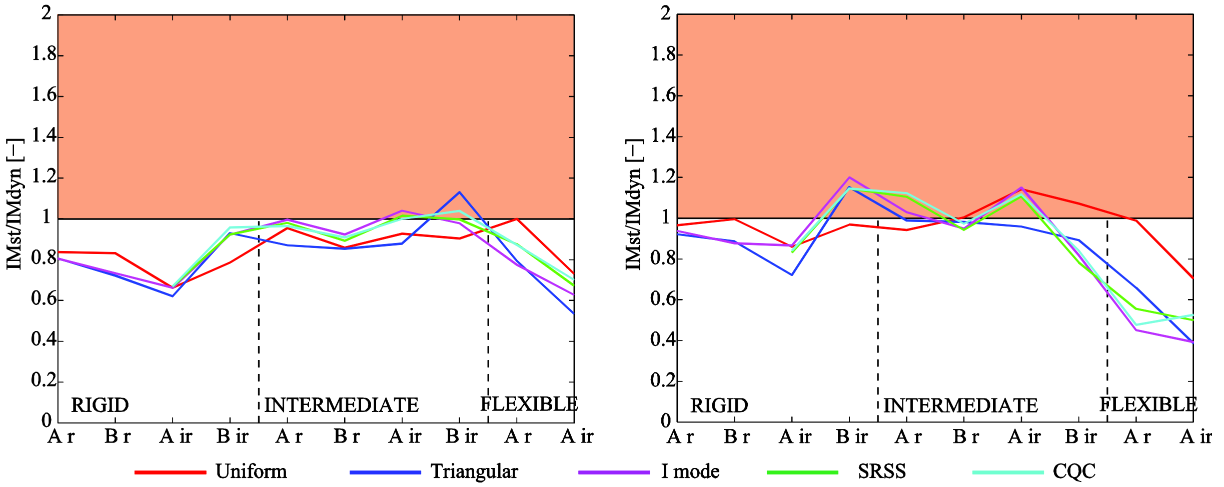

Figure 6. Damage limitation limit state, multiscale approach, ratio IM_{st}/IM_{din}. Left calculated with CSM, right with N2 method.

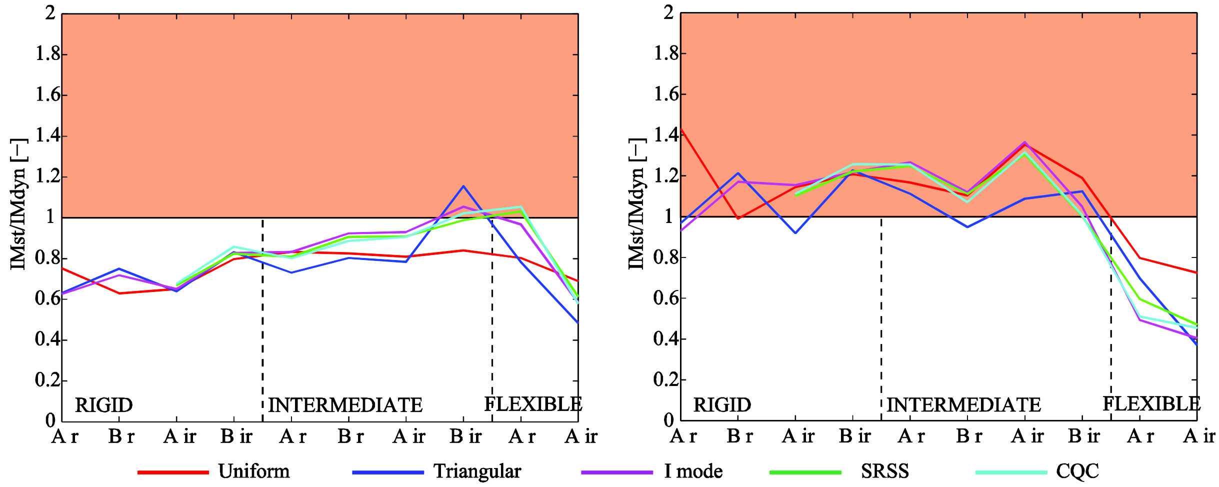

Figure 7. Life safety limit state, multiscale approach, ratio IM_{st}/IM_{din}. Left calculated with CSM, right with N2 method.

Probably the limit on the N2 method consists in being developed for the design of new buildings and for low levels of ductility, thus less reliable in case of the assessment of existing ones. Furthermore, this method is strongly influenced by the definition of the equivalent period T^*, that is strongly associated to the elastic response, while the overdamped spectra refer to the use of secant periods.

CONCLUSIONS

From the analysis of the results on the 10 models analyzed it is possible to observe:

- The non linear static analysis provides reliable results even if applied to plan irregular buildings with flexible diaphragms under the hypothesis to apply at least two load patterns. At this point of the research the authors suggest to use the *uniform* and the SRSS load patterns.
- It is suggested to use the CSM instead of the N2 method both because it provides more reliable and conservative results and because it does not require to transform the pushover curve in bi-linear relationship; therefore it is not strongly dependent by the definition of the period T^*.

- It is suggested an integration of the criteria defined in the codes for the definition of the performance levels with the criteria proposed by the multiscale approach (already codified in the Italian standard CNR DT 212 2013), especially in case of buildings with flexible diaphragms.

It is worth underlining however, that the results and the subsequent observations refer to models that present isolated URM buildings with flexible diaphragms and with plan irregularity. Possible refinements of the procedure could arise after analysis already scheduled on models that present elevation irregularity and/or that are representative of aggregate buildings.

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