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SEISMIC ASSESSMENT OF SINGLE-BLOCK ROCKING ELEMENTS IN MASONRY STRUCTURES

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Abstract

The paper focuses on the seismic assessment of single-block rocking elements in masonry structures, prone to rocking and overturning failure. They include both free-standing elements in ordinary or monumental buildings (such as parapets, battlements of fortresses, soaring portion of church façades), and single artistic artefacts (as statues, pinnacles or balustrades). As demonstrated by recent and past earthquakes, they are all affected by a significant seismic vulnerability, due to their high slenderness and small size (thickness). Since the use on nonlinear dynamic analyses is rather demanding and not practice-oriented, despite the complexity of nonlinear dynamics of rocking systems, different seismic assessment procedures have been proposed within the ambit of the Displacement-Based Approach (DBA). DBA methods have been sometimes already adopted in Standards, but a systematic validation of their results have not been provided yet. The latter motivates the focus of this paper, that is the reliability assessment of four DBA methods available in literature and standards, that are: New Zealand Guidelines for Assessment and Improvement of Existing Buildings, 2006; its updated version as issued in April 2015; Italian Technical Building Code, 2008; a recent paper by Lagomarsino, 2015. To pursue the validation aim, three different blocks were analysed: by exploring various size and slenderness, they have been conceived to be representative of typical masonry elements prone to rocking like pinnacles, parapets or plane-belfries. Incremental Dynamic Analyses (IDA) have been performed by using several real accelerograms. Their results, in terms of Peak Ground Acceleration leading to overturning of the block, were considered as reference solution, to be compared with the ones obtained by applying the considered DBA procedures. These latter differ in the assumed ultimate displacement capacity and the equivalent period to be used for the evaluation of the displacement demand from the acceleration-displacement response spectrum. The three blocks were assumed alternately resting on the ground floor or at the top of a masonry main building. The comparison in terms of fragility curves allowed to evaluate safety margins, robustness and reliability guaranteed by the static previsions. The results of the performed analyses (which could be expanded in the future by considering further case studies and ground motions) showed that not all the proposals present in literature are equivalent and, in particular for two of them, confirmed the potential reliability of the equivalent static approach instead of a more demanding dynamic one for the seismic assessment of rocking elements. In fact, under proper definitions of the ultimate displacement capacity and of the procedure to evaluate the seismic displacement demand, the DBA approach turned out reliable for both ground and atop rocking elements. Moreover, the performed nonlinear dynamic analyses clearly highlighted that the displacement demand is very sensitive to the intrinsic features of each record and the assessment cannot be made by assuming only a mean spectrum (as usually adopted in Standards). To this aim, a new proposal is currently underway to correct the mean spectrum and improve the reliability of the selected DBA procedure.

Keywords: rocking; displacement-based assessment; single-block elements

1. Introduction

The observation of damage caused by recent and past earthquakes has demonstrated the high seismic vulnerability of historic masonry structures, especially referring to the activation of out-of-plane mechanisms in masonry walls and rocking response of stone or masonry elements (Fig.1). These latter include not only free-standing masonry elements in ordinary and monumental buildings (such as parapets, battlements of fortresses, soaring portion of church façades), as well as stone artistic artefacts (as statues, pinnacles or balustrades), all characterized by an intrinsic seismic vulnerability, due to the high slenderness and the small size (thickness). These evidences are documented for example in [1, 2].

Fig. 1 – Recurrent mechanism involving the overturning of rocking elements (from the Emilia Earthquake, 2012)

The complexity of nonlinear dynamics of rocking systems promoted research aimed to understand the peculiarities of the rocking dynamic response of rigid bodies [3, 4, 5]. However, all these studies, presumed to solve the equations of motion; hence, they never found place into Standards, since they implied the use of demanding and rather complex tools. For this reason, at the same time, in order to implement more practice-oriented tools, other Authors [6, 7, 8] developed in literature proposals within the ambit of the Displacement-Based Approach (DBA). Such seismic assessment procedures have been already adopted, sometimes in a simplified manner, into International Standards, but a systematic validation and comparison of the provided results have not been provided yet. Furthermore, many Authors [5] raised doubts on the actual reliability of DBA procedures due the inherent dynamic instabilities and chaotic behaviour characterizing rocking response.

In this framework, the paper aims to verify the reliability in performing the seismic assessment of both atop and ground elements of procedures available in international Standards and based on the equivalent static approach. With the aim of validation, Incremental Dynamic Analyses (IDA) have been performed by considering several real accelerograms and different blocks. The blocks have been alternatively assumed resting on the ground floor or in an atop position. The results of the IDA have been considered as reference solution and then compared with those obtained by applying four different DBA methods. Finally, the results of the validation were statistically treated.

2. Out-of-plane assessment in Standards

The procedures considered in the validation have been selected as the most advanced in literature and were taken from the following Standards or research papers: New Zealand Guidelines for Assessment and Improvement of Existing Buildings, 2006 and its updated version as issued in April 2015 [9]; Italian Technical Building Code, 2008 [10]; 4) a recent paper by Lagomarsino, 2015 [8]. Hereafter, it is referred to them respectively as: NZ-old; NZ-new; NTC08 and L-2015. In particular, the NZ standards are founded on the theoretical framework of [7]. The main differences and analogies among the examined procedures are summarized below.

In general, a DBA method consists in a proper comparison between the system capacity and the seismic input expressed in terms of response spectrum. In this comparison, two are the main aspects involved and which have been the main subject of the validation: i) the definition of the ultimate displacement capacity d_u, which represents a cautionary displacement such to be enough far from the complete block overturning condition; ii) the definition of a procedure to compute the displacement demand corresponding to the ultimate condition.

Referring to the four examined procedures, Fig.2 highlights differences and analogies in the definition of these latter aspects. In particular, the figure illustrates how the four procedures can be applied with the aim of calculating the value of the maximum Peak Ground Acceleration (PGA) compatible with the achievement of the ultimate displacement capacity of the system assumed as reference. Hence, the PGA is calculated by imposing the ultimate displacement capacity equal to the displacement demand computed through T_s. In the figure the response spectra are consistently scaled to the different PGA values obtained. It has to be specified that the two capacity curves (in orange in the figure) reflect the fact that, even if all the methods are based on the same formulation for the substitute structure [11] and dynamically the oscillator is the same, they refer to the displacement of different points of the block: in fact, the New Zealand Guidelines consider the displacement of a point at 2/3 of the panel height, while NTC08 and L-2015 consider the displacement of a point at half height.

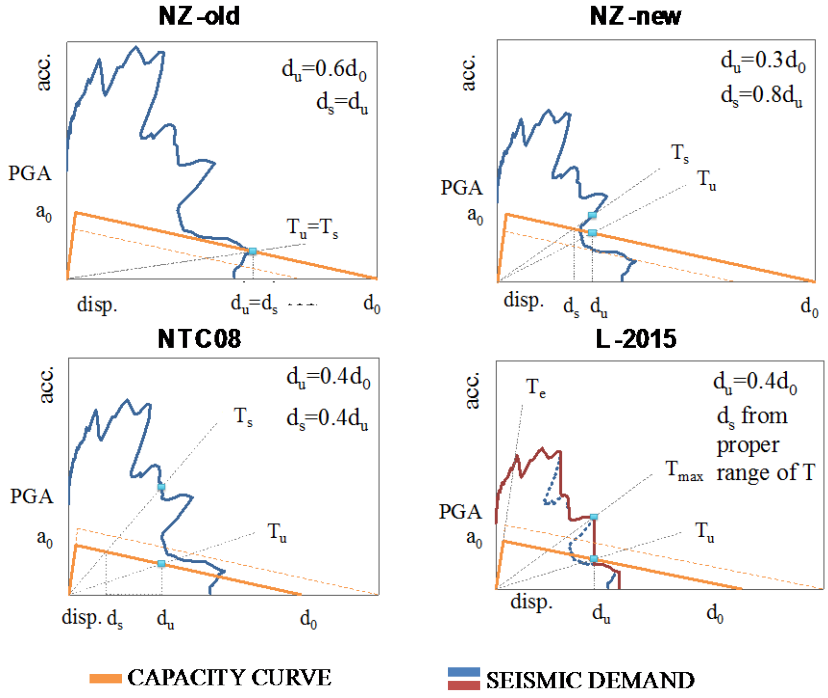

Fig. 2 – Sketch of the application of the examined procedures

From Fig.2, it is possible to see that:

i) in the definition of the ultimate displacement capacity d_u, all the four procedures generally consider a fraction of the displacement d_0, which corresponds to the block collapse for loss of static equilibrium. Then, depending on the examined procedure, the threshold assumed can be more or less precautionary;

ii) in the evaluation of the displacement demand, the theoretical differences are more significant. The first one concerns the response spectrum adopted. In the NZ guidelines and in NTC08 it is the elastic one, eventually amplified, but however not modified. Instead, in L-2015 the response spectrum is before made smooth in order to remove all the indentations which usually characterize response spectra of real records (see the red line in Fig.2). In this way, in such modified response spectrum, the spectral displacement is never reduced by increasing the period T; moreover, even in a simplified manner, it implicitly takes into account the peaks present in the response spectra which could affect the dynamic response of rocking systems. The second difference concerns the way to compute the displacement demand. While all codes procedures prescribe a pre-fixed value for the reference period T_s (adopted by considering a displacement d_S which is a fraction of the displacement capacity), in L-2015 the displacement demand is obtained by a direct intersection through the capacity curve and the modified response spectrum.

3. Selection of case-studies and input data

The rocking and overturning of a single block, represented by a vertical cantilever, has been herein considered. The IDA were performed by considering 648 real accelerograms, compatible with a seismic action of L'Aquila, selected within the framework of ReLUIS/EUCENTRE Project "Seismic risk of structures implied by design in Italy", coordinated by Iunio Iervolino and Paolo Bazzurro. Starting from such accelerograms, for the application of the DBA procedures, the corresponding response spectra (or floor response spectra) have been directly generated through a step-by-step integration of the records (filtered for atop blocks) considered as input. This to avoid the influence of how the floor spectrum was determined, which is different from standard and standard and not always able to properly describe the amplification phenomenon and the effects on it connected to the nonlinearities of the main supporting structure [12, 13]. A 3-DOF system representative of an ordinary masonry building was used to filter the seismic input. The 3-DOF system was considered alternately linear elastic and nonlinear; in this latter case, an equivalent elastic model with a damping equivalent to increasing levels of ductility μ was considered (Fig.3).

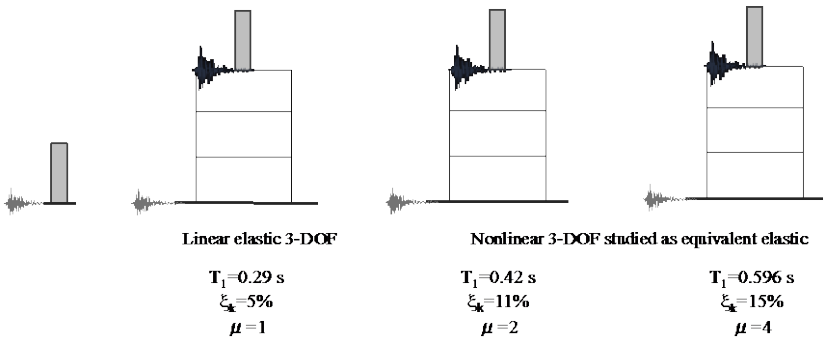

Fig. 3 – Blocks alternatively considered in the validation resting on the ground floor or standing at the top of the 3-DOF system, representative of the main structure

Three blocks of various geometries have been considered in order to be representative of different types of standing masonry elements prone to rocking (parapets, chimneys, soaring portion of façade, spires, battlements). Table 1 illustrates the characteristics of the three considered blocks, in terms of: slenderness (λ); thickness (2b); height (2h); elastic period (T_e); damping (ξ). Blocks have three values of slenderness (λ=5-3-7) and two of width (2b=0.22-0.44 m) in order to investigate the influence of size and shape. In particular, Panel 1 simulates a parapet, Panel 2 a pinnacle, while panel 3 a plane belfry.

Table 1 – Characteristics of the three considered blocks

Block	Typical Asset	λ [-]	2b [m]	2h [m]	T_e [s]	ξ □□□
1	Parapet	5	0.22	1.10	0.18	0.05
2	Pinnacle	3	0.22	0.66	0.015	0.08
3	Plane belfry	7	0.44	3.08	0.233	0.03

For the blocks, a bi-linear capacity model has been considered in the IDA. This latter choice was also supported by the results of the experimental campaign performed in the University of Genoa [14, 15], which highlighted that a bi-linear model describes better than the classical Housner model the rocking response of actual masonry panels. The bi-linear model is characterized by an initial elastic branch, defined by a pseudo-elastic period of vibration. This initial elastic branch is fundamental for the assessment of the seismic behaviour with respect to the damage limit state, being often in the range of maximum amplification of the acceleration

response spectra. As a consequence, the rocking activation takes place before PGA is equal to the static horizontal multiplier, since it is related to the spectral acceleration for the above-mentioned period.

4. Validation approach

The validation approach consisted in the following steps, repeated for each block geometry and varying the assumed position (on the ground floor or at the top of the 3-DOF system).

Step 1: Incremental Dynamic Analyses

Incremental Dynamic Analyses (IDA) have been performed by scaling PGA of each time history, from a very low value till to overturning conditions [16]. Each time history defines an IDA curve (Fig.4a). The IDA curve is represented on a graph which has on the x-axis a response variable monitoring the structural response to that ground motion (namely an Engineering Demand Parameter - EDP) and on the y-axis an Intensity Measure (IM). In the examined case, we assumed the maximum horizontal displacement at the block barycentre as EDP (named d_u hereafter) and the PGA as IM.

The choice of the Scale Factor (SF) to scale each accelerogram (in our analyses assumed constant and equal to 0.025g) is a crucial aspect. In fact, it has to be not too small in order to guarantee a bearable computational efforts of the analyses, but at the same time small enough to be sure to properly catch the onset of a dynamic instability phenomenon, sometimes occurring and characterized by a twisting pattern of the curve. In engineering terms a non-monotonic curve means that the block, once experienced a certain value of displacement induced by a certain PGA, may be subjected to a smaller displacement for increasing accelerations: this is due to the presence in the input record of an opposite impulse, which tends to straighten the block, instead of inducing a higher rotation.

By applying the same procedure to all selected time-histories, a set of IDA curves has been obtained for each block (Fig.4c). Such set has been then used to get a probabilistic representation of the seismic demand conditioned to IM (Fig.4d): in particular, at each level of IM, the median, 16% and 84% IDA curves have been computed by assuming a lognormal distribution. For a given level of IM, such statistical evaluation has been performed by considering also the number of the so called “certain collapses”, which can be determined once defined for each time history the IM value that produces the collapse condition.

Fig.4 summarizes the main steps of the performed IDA. The construction of the IDA curve set is presented from Fig.4a to Fig.4c, while in Fig.4d, the median, 16% and 84% IDA curves are plotted in yellow.

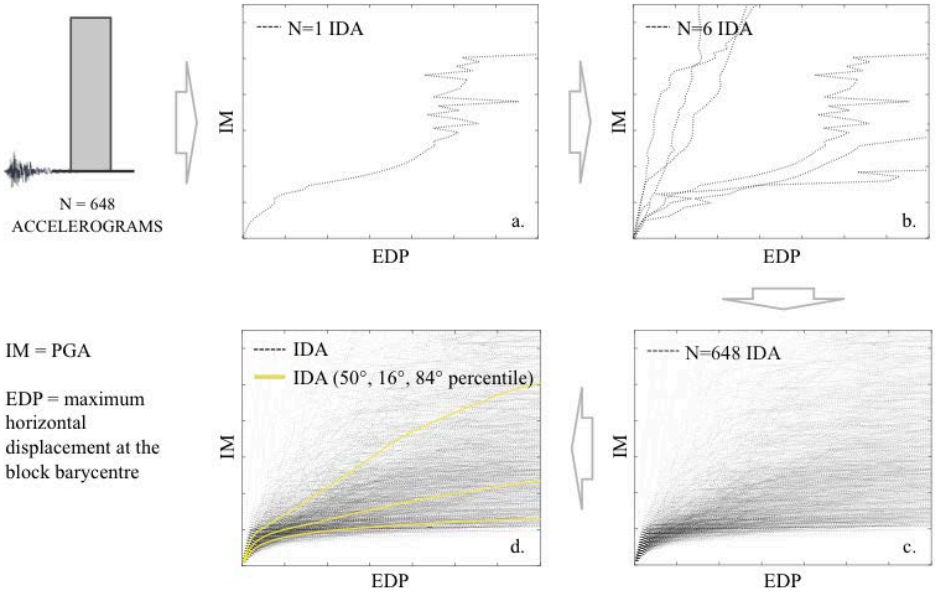

Fig. 4 – Sketch of the steps performed in the IDA

From the IDA, it has been possible to obtain:

i) the values of PGA corresponding to the actual dynamic collapse, hereafter named as PGA_{IDA};

ii) the values of the actual PGA corresponding to the achievement of the limit condition assumed by codes, hereafter named as $PGA_{IDA,Code}$; in particular, these latter have been determined by entering in the IDA curves with the EDP values corresponding to the ultimate displacement capacity assumed by the four static procedures, hereafter named as $d_{u,Code}$.

Step 2: Application of Displacement-Based Approach (DBA) procedures

For each block geometry and related position (on the ground floor or atop), the four examined DBA procedures have been applied and then the results have been then compared with the ones of the IDA, assumed as "reference solution". In particular:

i) by applying each procedure using the *N* response spectra generated from the *N* accelerograms, we determined *N* values of PGA inducing the ultimate displacement capacity $d_{u,Code}$, named hereafter as PGA_{Code}; then, these values have been statistically treated, by evaluating their median, 16% and 84%;

ii) by applying each procedure using the median, 16% and 84% response spectra obtained from the *N* considered accelerograms, we determined three values of PGA for each procedure, named in hereafter respectively as $PGA_{Code,50}$, $PGA_{Code,16}$ and $PGA_{Code,84}$.

These latter values are equal to the fractile values evaluated from the PGA_{Code} for the methods of NZ-old, NZ-new and NTC08, while they are different for L-2015; this is due to the modification of the response spectrum suggested in [8], aimed to remove all the indentations and that consequentially determines a no decreasing displacement response spectrum. Therefore, the median and fractile response spectra obtained from the modified record-to-record response spectra are different from the response spectra obtained as in (ii). Hereafter, the fractile values of the PGA obtained with L-2015 applied record-to-record and then statistically treated will be named as $PGA_{L\text{-}2015,50*}$, $PGA_{L\text{-}2015,16*}$ and $PGA_{L\text{-}2015,84*}$ (hence, the asterisk indicates that the L-2015 procedure has been applied record-to-record).

5. Comparison between IDA and DBA procedures

In order to guarantee a robust validation of the DBA procedures in terms of both reliability and safety, different kinds of representation and comparison among results from IDA and DBA methods have been provided. Herein only the main obtained results are presented, while a more detailed description is illustrated in [17].

5.1 Reliability of the examined DBA procedures

In order to express a judgment in terms of reliability, a comparison in terms of cumulative curves is firstly presented. The cumulative curves have been computed by organizing in an increasing order the ratio between PGA_{Code} and $PGA_{IDA,Code}$ as obtained by record-to-record. In particular, the median value of these cumulative curves gives a measure of how the examined equivalent static method is capable in average to predict the PGA corresponding to the achievement of the actual ultimate displacement capacity $d_{u,Code}$ as obtained by the IDA, while the curve slope (which is the beta value of the lognormal distribution) indicates how random this prevision is. The most reliable static method would have a cumulative curve almost vertical and as much as possible close to 1 (Fig.5a). The curve will be much more smaller than 1 as the static prevision is more precautionary (e.g. curves no.1 and no.2 in Fig.5b). If a method has a median value of $PGA_{Code}/PGA_{IDA,Code}$ smaller than 1 and beta small, the method is precautionary and robust (e.g. curve no.1 in Fig.5b); on the contrary, if the median is close to 1 but beta is high, it means that the method works on average, but in a random way (e.g. curve no. 3 in Fig.5b).

Fig.5 illustrates some examples of cumulative curves: in particular, the green field indicates where the prevision of the DBA method is more precautionary.

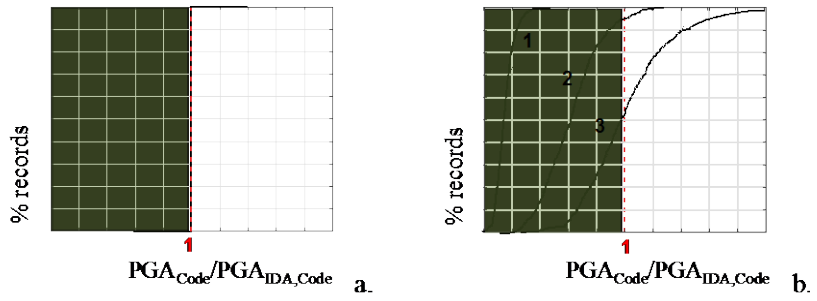

Fig. 5 – Examples of cumulative curves: a) ideal curve, representative of the most reliable static method; b) actual cumulative curves, providing previsions more or less precautionary and scatter

Fig.6 shows the cumulative curves obtained for example for block 1, alternately considered on the ground floor, at the top of the elastic 3-DOF system (3-DOF – μ=1) or at the top of the nonlinear system (3-DOF – μ=2; 3-DOF – μ=4). Similar trends have been obtained for the other two considered blocks [17]. From the cumulative curves, it is possible to observe that the NZ-old is the methodology which systematically is characterized by the highest dispersion, while L-2015* is always the most precautionary. Furthermore, in general the most reliable methods are the ones proposed in L-2015* and in NZ-new.

However, comparing the cumulative curves obtained applying these latter procedures, varying the three block geometries and position, it emerged that:

i) the procedure proposed by L-2015* works better, considering the robustness of the reliability of the method by varying the type of panel: in fact, it is able to guarantee the most precise and stable prevision, with respect to the dynamic results;

ii) the procedure proposed in NZ-new works worse for the atop blocks.

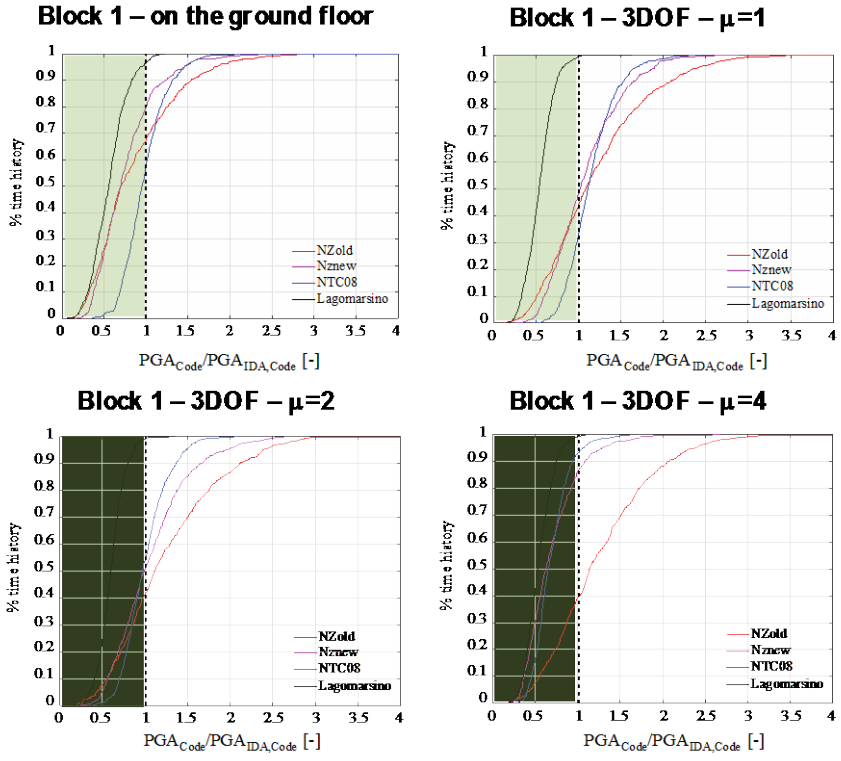

Fig. 6 – Cumulative curves for block 1

Fig.7 shows for example the cumulative curves for the three blocks, resting on the ground floor, for the procedure of L-2015* and NZ-new. One can see that NZ-new works worse: for example, for block 3 there is a certain number of cases where the prevision provided by the method is significantly not precautionary.

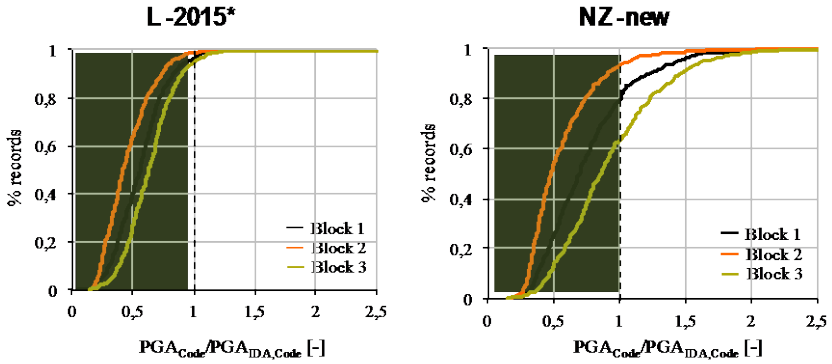

Fig. 7 – Cumulative curves for the three blocks, resting on the ground floor for the procedure of L-2015* and NZ-new

5.2 Failure probability assessment

In order to evaluate the safety margins guaranteed by each procedure, the comparison among DBA procedures is then presented in terms of failure probability, evaluated by a fully probabilistic approach. To this aim, the classical tools of the seismic reliability analysis are used [18] by referring in particular to the basic reliability "resistance R" – "load effect S" model adopted in the Load Resistant Function Design. As known, in general terms, the failure probability P_f can be obtained by introducing a proper variable G that expresses the limit-state function and by defining the subspace of the structure's unsafe or failure states. A quite common assumption for the variable G is to be expressed as the difference (R-S), often denoted as safety margin. Under the hypothesis that R and S are independent random variables and Gaussian, it is possible to apply the properties of linear combination of Gaussian variables and P_f is given in a closed form as:

$$P_f = P(R - S \le 0) = \Phi\left(\frac{-(\mu_R - \mu_S)}{\sqrt{\sigma_R^2 + \sigma_S^2}}\right) \quad (1)$$

where $\mu_{R(S)}$ and $\sigma_{R(S)}$ express the mean and standard deviation of R and S, respectively, and $\Phi(\cdot)$ is the standard Gaussian Cumulative Distribution Function (CDF). From Eq. (1) it is evident that the probability of failure increases when the variability of either load effect or resistance increases (i.e. σ_R or σ_S, or both), or when the margin between resistance and load effect means reduces. In the context of probabilistic assessment, First Order Reliability Methods (FORM) arise promoting the use of indices as simple reliability measures for whose computation of first or second moment characterization of the random variables involved suffice. In particular, it has been proposed by [19] to take as second moment reliability index the ratio between the mean value and the standard deviation of G that, under the hypothesis of Gaussian function, leads to:

$$P_f = \Phi\left(-\frac{\mu_G}{\sigma_G}\right) = \Phi\left(-\beta_C\right) \quad (2)$$

which establishes a biunivocal relationship between the Cornell index and the probability of failure. According to Eq. (2), it is convenient to assume R and S as lognormal distributed (since as known the properties of the latter is the logarithm of the function is distributed as a Gaussian) and express the limit state function G as the nonlinear function given by the ln(R/S). It is easy to demonstrate, by applying the logarithm properties, that the

latter is equivalent to ln(R) –ln(S), that is what already introduced in Eq. (1). In the following for the computation of P_f reference is made to the latter assumption - that is Eq. (2) and G defined as ln(R/S).

Hence, to apply the aforementioned concept, it is necessary to establish a parallelism between the classical model "resistance R - load effect S" and the model "capacity C – demand D", which is the one usually adopted in the seismic framework. In particular, in the examined case:

i) the resistance R corresponds to the actual system capacity C which is herein represented by the maximum Intensity Measure obtained from IDA, corresponding to the actual dynamic overturning of the block (PGA_{IDA});

ii) the load effect S corresponds instead to the demand D which is represented by the maximum Intensity Measure computed according to each DBA method as compatible with the achievement of the ultimate displacement capacity $d_{u,Code}$ (PGA_{Code}).

It is important to observe that in the examined case the capacity C and the demand D cannot be assumed as statistically independent since in the application of the DBA methods the response spectra directly generated from the same records used for the IDA have been adopted. Thus the use of Eq. (2) makes the computation of P_f even more convenient since the standard deviation is computed directly on the variable ln(R/S), thus taking into account the correct dispersion, which is significantly lower than that estimated by assuming C and D as uncorrelated.

Fig.8 presents in a logarithmic scale for the three DBA methods working better (as resulting from §5.1), the obtained failure probabilities, varying the block geometry and position. In particular, the different examined conditions (block on the ground floor or atop) are on the x-axis, while the failure probabilities are on the y-axis. From Fig.8 it is expected that the most reliable, safe and stable method would have values of P_f low enough and as constant as possible, varying the block geometry and position.

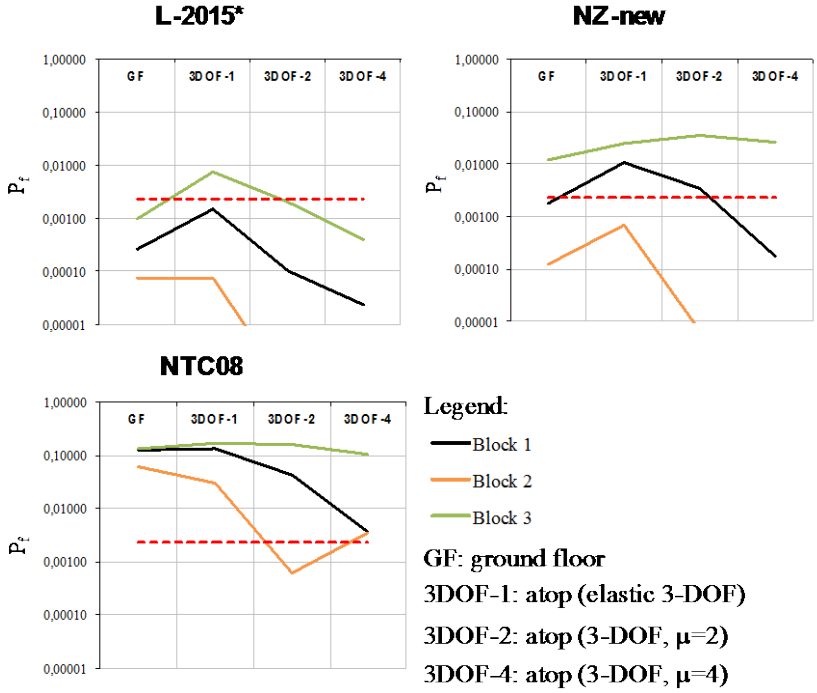

Fig. 8 – Trend of the failure probabilities obtained with three DBA procedures, varying block geometry and position

In order to evaluate if the safety margins guaranteed by each procedure were sufficient, it was useful to define a minimum threshold with which these values can be compared. In the examined case, the minimum

levels of protection in terms of mean annual frequency of exceedance (λ_{LS}) suggested in [20] have been assumed as reference. In particular, the threshold equal to 0.0023 has been selected that refers to Class II (ordinary buildings) and the limit state SLC (corresponding to a situation where the building is still standing but would not survive an aftershock); the latter is assumed equivalent to Near Collapse conditions. This threshold is identified in Fig.8 by the dashed red line. Of course, in real applications, different reference values could be defined as a function of the type of examined asset.

From Fig.8, it is possible to see that the less reliable and safe method is that prescribed in NTC08, while the procedures of NZ-new and of L-2015* work better. However, while the P_f calculated with the method of NZ-new are in many cases higher than the considered conventional threshold, in the method of L-2015* the obtained probability failures is always lower, except for the case of block 3 placed at the top of the elastic 3-DOF system.

6. Conclusions

The results of the analyses (performed up until now by considering specific blocks' geometries and ground motions, and which could be in the future expanded by considering further case studies and inputs) showed in general the reliability of DBA methods. In particular, the results of validation, presented in terms of comparison among cumulative curves (§5.1) and failure probability assessment (§5.2), highlighted that the DBA provides a coherent and reliable estimate of overturning of masonry element subjected to rocking, with the aim of checking the Near Collapse Limit State, in particular if applied in the form proposed by L-2015, applied record-to-record.

This has been proved by the following results:

i) the method is the most accurate in evaluating PGA that produces a displacement demand equal to the ultimate displacement capacity (PGA_{L-2015}): the cumulative distribution of the reliability ratio (Fig.6 and Fig.7) always lower than one and with the lowest dispersion, if compared with the other methods;

ii) the method, if applied record-to-record from the response spectra of real accelerograms, turned out to be the only one able to guarantee values of the probability of failure (evaluated by a fully probabilistic approach, by considering as limit state function the logarithm of the ratio between demand and capacity, in terms of PGA) rather stable for the different cases (Fig.8) and compatible with the one adopted for Near Collapse Limit State in the case of seismic actions [20].

A drawback in the application of L-2015* method is that, at engineering practice level, the seismic input is defined by code response spectra or, at least, by conditional mean spectra obtained by a specific Probabilistic Seismic Hazard Assessment (PSHA). These smooth spectra are not able to properly take into account the significant features of each single record, related to single or multiple peaks which are described by the correspondent acceleration-displacement response spectrum and taken into account by the specific regularization proposed by the method (transformation into a no decreasing displacement response spectrum).

Therefore, two alternative options can be adopted for the practice-oriented use of the L-2015* method within the ambit of the seismic assessment of rocking masonry elements, once the seismic input is provided by a smooth response spectrum:

i) use of the DBA method record-to-record, by considering the selection of a suitable number of records (e.g. between 20 to 30) compatible with the input spectrum, taking into account the relevant seismological features in the site, assumed for the PSHA in terms of magnitude, distance and focal mechanisms;

ii) application of a corrective amplification factor to the input response spectrum obtained from the PSHA, in order to implicitly take into account the transformation that would be necessary to make record-to-record. This latter proposal it is currently underway. For more details, the authors refer to [17].

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