Non-linear Static Analysis of Masonry Buildings under Seismic Actions

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ABSTRACT

Assessing existing masonry buildings in seismic zones is a critical issue, due to the high vulnerability of the built environment. Commonly, beside local analysis, refined assessment procedures are used based on non-linear static methods, like the pushover method, supplemented, if necessary, by dynamic linear analyses, devoted to check the order of magnitude of the results. In non-linear static analysis masonry buildings are mostly modelled using the so-called equivalent frame model, but the resulting structural scheme is usually very complicated and the analysis requires to be "driven" step by step by very skilled users in order to obtain consistent results.
An innovative and "robust" method is proposed for non-linear static analysis of masonry building. The method, which relies on very simple structural models, nearly independent on the user, recovers some basic assumptions of the classical POR method, and can be applied to mono or multi-story masonry buildings. Comparing the results obtained with the proposed method with those derived using the classical pushover analysis in several relevant case studies, allowed to validate it. Moreover, the practical applications confirmed that the method is suitable for refined assessment of the seismic performance of the structure, with a limited computational effort, so making possible also extensive sensitivity studies.

Keywords: Masonry Buildings, Seismic Assessment, Seismic Risk Index, Non-linear static analysis, Pushover Method.

1. INTRODUCTION

Large areas of historical towns and more generally the built environment are very sensitive to seismic actions; in effect, they are constituted by ancient masonry constructions built according traditional empirical rules, disregarding any specific seismic provisions, and, in any case, with no direct consideration of seismic actions.

The definition of suitable procedures for the assessment of seismic vulnerability of existing relevant masonry buildings becomes thus very important, also considering that these procedures could be applied for new buildings too.

Commonly, beside local analysis, refined assessment procedures are used based on non-linear static methods, like the pushover method, supplemented, if necessary, by dynamic linear analyses, mainly devoted to check the order of magnitude of the results. In non-linear static analysis masonry byuildings are mostly modelled by means the so-called equivalent frame model, but the resulting structural scheme is usually very complicated and the analysis requires to be "driven" step by step in order to obtain consistent results.

In the past, a simplified and effective method for the seismic resistance verification of unreinforced masonry buildings named POR, was firstly introduced by Tomažević after the Friuli (I) earthquake in 1976 [1] [2], and adopted by the Italian regulations published in 1978 [3] and 1981 [4] for the reparation of existing masonry buildings. The method, which was one of the first computer programmes regarding the seismic assessment of masonry building, can be seen as a simplified non-linear, pushover type method [5]. It hypothesizes a story failure mechanism [6] for the whole building, where the global response of each story is evaluated in terms of base shear and story displacement considering the sum of the individual response of each wall: The shear response of each resisting wall is modelled via an elastic-plastic constitutive law with a limited plastic plateau, or, in other words, limited ductility. The shear resistance curves for each story of the investigated building are calculated assuming that the maximum shear capacity of the story itself corresponds to the first attainment of the ultimate displacement in a wall. The seismic assessment is fulfilled when the shear resistance is bigger than the design seismic shear.

The POR method considers each story separately, so it allows to evaluate the maximum shear resistance of individual floors, but it disregards the ductility of the whole structure; moreover, even if very efficient from the computational point of view, it presents some inaccuracies [7] and limitations.

Furthermore, performing the verification in terms of shear resistance, it is not in line with the most updated assessment techniques adopted in pushover analysis, where seismic capacity and demand are evaluated using the Acceleration Displacement Response Spectra (ADRS) diagram.

In the paper, an innovative and "robust" non-linear method for non-linear static analysis of masonry building is proposed, which, keeping the simplicity and the computational efficiency of the POR method, removes its limitations and inaccuracy.

The method allows to take into account the ductility of the whole structure and can be applied to analyze mono or multi-story buildings, where stiffness of the floors can be, in turn, zero or infinity, depending on the characteristics of the floors themselves.

The authors, belonging to the research unit of the University of Pisa, convened by the corresponding author, developed the present work in the framework of a research agreement between the Municipality of Florence and the Department of Civil and Industrial Engineering of the University of Pisa, concerning the assessment of seismic vulnerability of the school masonry buildings owned by the Municipality of Florence, aiming to provide a reliable and expeditious methodology for the seismic assessment of masonry buildings, also in view of prioritizing strengthening intervention.

2. BASIC ASSUMPTIONS

In the proposed method shear walls in the relevant directions *x* and *y* are considered in the analysis.

A shear wall is able to transfer horizontal or seismic forces to the soil, so it corresponds to a bearing wall, characterized by vertically aligned masonry walls, continuous to the foundation. According to this definition, in a *N*-story building a shear wall can be effective for *j* floors, *j*<*N*, provided it is continuous from the *k*th floor to the foundation. On the contrary, influence and modeling of walls not extended to the foundation, if any, cannot be considered in a general way and should be preliminarily studied case by case, since they represent a formidable source of structural irregularity and local vulnerability.

Assuming both ends clamped, the lateral stiffness k of a masonry wall can be calculated, taking into account shear deformations and bending effects [5], as

$$k = \frac{GA}{1.2h}\left(1 + \frac{G}{1.2E}\left(\frac{h}{l}\right)^2\right)^{-1}, \qquad (1)$$

where h is the inter-story height of the wall, l its length, A the area of its cross-section, and E and G are the modulus of elasticity and the shear modulus of masonry respectively.

It must be stressed that mechanical properties of masonry to be introduced in Eq. (1), especially the shear modulus, influence significantly the outcomes of non-linear analysis of masonry buildings, so that their sound evaluation represents a crucial issue in the seismic assessment. Since this topic is outside the scope of the present work, it will not be discussed further.

If diagonal tension shear failure governs, the shear resistance of the wall H_{Rd} can be derived from

$$H_{Rd} = A\frac{1.5\tau_{0d}}{b}\sqrt{1 + \frac{\sigma_0}{1.5\tau_{0d}}} \qquad (2)$$

where σ_0 is the compressive stress induced in the wall in the seismic combination, τ_{0d} is the shear strength of masonry and b is the shear resistance factor, depending on the aspect ratio h/l of the wall. When $h/l \geq 1.5$, it can be assumed $b = 1.5$ [5].

The hysteretic behavior of the masonry walls subjected to constant vertical load and cyclic horizontal loads is idealized means of a bilinear resistance envelope like in the original POR program [1] [2]. The bilinear envelope is characterized by an initial elastic slope, defined by the lateral stiffness k, and by a plastic plateau, limited by the elastic inter-story drift δ_e and by the ultimate inter-story drift δ_u, corresponding to the shear resistance of the wall H_{Rd}. Obviously, combining Eq. (1) and Eq. (2), it results

$$\delta_e = \frac{H_{Rd}}{k} = 1.2\frac{\tau_{0d}}{G}h\left(1 + \frac{G}{1.2E}\left(\frac{h}{l}\right)^2\right)\sqrt{1 + \frac{\sigma_0}{1.5\tau_{0d}}}, \qquad (3)$$

while the ultimate displacement can be obtained as

$$\delta_u = \mu\delta_e, \qquad (4)$$

being μ the ductility factor. For unreinforced masonry, in [4] it is recommended to assume $\mu = 1.5$.

Obviously, the ductility of unreinforced masonry is not a ductility in a conventional sense [8], but rather the relative slip along crack surfaces that parts of the wall elements can sustain without significant shear stress loss; it depends on normal stress σ_0 [8] [9], on wall geometry via the aspect ratio h/l and on boundary conditions [10].

In current versions of Eurocode 8 [11] as well as of the Italian Building Code [12] [13] values of ductility factors are not given, being the ultimate drift defined as a percentage of the inter-story height of the wall, depending on the failure mode. For shear failure, the ultimate drift is set to

$$\delta_u = 0.4\%\, h\,. \qquad (5)$$

In the implementation of the proposed method, described in §3, it is possible to choose one of the two abovementioned options, so that it is possible to define ultimate drift in terms of ductility or in terms of percentage of the inter-story height.

3. THE E-PUSH PROGRAM

With the basic assumptions already recalled, a pushover type algorithm has been implemented, as described below.
The algorithm, called E-PUSH, is in the author's opinion an enhancement of the classical pushover programs for masonry buildings, especially in terms of easiness and simplicity of modelling, in terms of speed of elaboration, not particularly demanding in terms of skill of users. An example of 3-D model of a two-story masonry building is reported in Fig. 1.

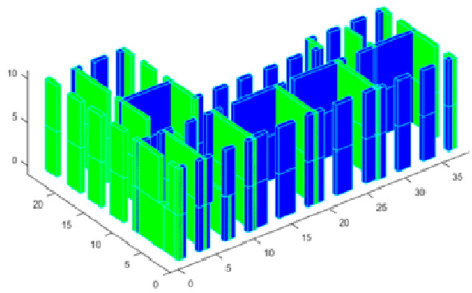

Fig. 1. E-PUSH 3-D model of a two story masonry building

In the following, only the case of infinitely rigid floors is considered, but the algorithm can be easily adapted to the case of floors that are very deformable in the horizontal plane, considering that analysis can be limited to aligned shear walls, subjected to loads coming from the adjacent areas.

Considering a *N*-story masonry building, characterized by *n* shear walls in *x* direction and *m* shear walls in *y* direction, the algorithm is summarized in the flowchart represented in Fig. 2.

The first step of the procedure is the calculation of the coordinates of the center of mass, $\overline{x_{M,k}}$ and $\overline{y_{M,k}}$, and of the center of rigidity, $\overline{x_{R,k}}$ and $\overline{y_{R,k}}$, for each story *k*:

$$\overline{x_{M,k}} = \frac{\sum_{i=1}^{n+m}\sigma_{0i}A_i x_i}{\sum_{i=1}^{n+m}\sigma_{0i}A_i}; \quad \overline{y_{M,k}} = \frac{\sum_{i=1}^{n+m}\sigma_{0i}A_i y_i}{\sum_{i=1}^{n+m}\sigma_{0i}A_i}; \qquad (6)$$

$$\overline{x_{R,k}} = \frac{\sum_{i=1}^{n} k_{y,i} x_i}{K_{y,k}}; \quad \overline{y_{R,k}} = \frac{\sum_{l=1}^{m} k_{x,l} y}{K_{x,k}}; \tag{7}$$

being

$$K_{y,k} = \sum_{i=1}^{n} k_{y,i,k}\,; \quad K_{x,k} = \sum_{l=1}^{m} k_{x,l,k}. \tag{8}$$

and the sums extended to all the shear walls present at the considered floor. Accordingly, the eccentricities of the story are

$$e_{x,k} = \overline{x_{M,k}} - \overline{x_{R,k}};\; e_{y,k} = \overline{y_{M,k}} - \overline{y_{R,k}}; \tag{9}$$

and the polar moment of inertia of the stiffness $J_{R,k}$ is

$$J_{R,k} = \sum_{i=1}^{n} k_{y,i}\left(x_i - x_{R,k}\right)^2 + \sum_{l=1}^{m} k_{x,l}\left(y_l - y_{R,k}\right)^2. \tag{10}$$

Then, a suitable distribution of seismic forces at each story is considered, which is increased at each step. For the sake of simplicity, in the flowchart in Fig. 2 a distribution of forces proportional to the elevation h_k of the floor is considered, according to the equation

$$F_k = F_h \frac{W_k h_k}{\sum_i W_i h_i}, \tag{11}$$

where F_h represents the total shear force at the base of the building, but it clearly applies also to different distributions of seismic or horizontal forces.

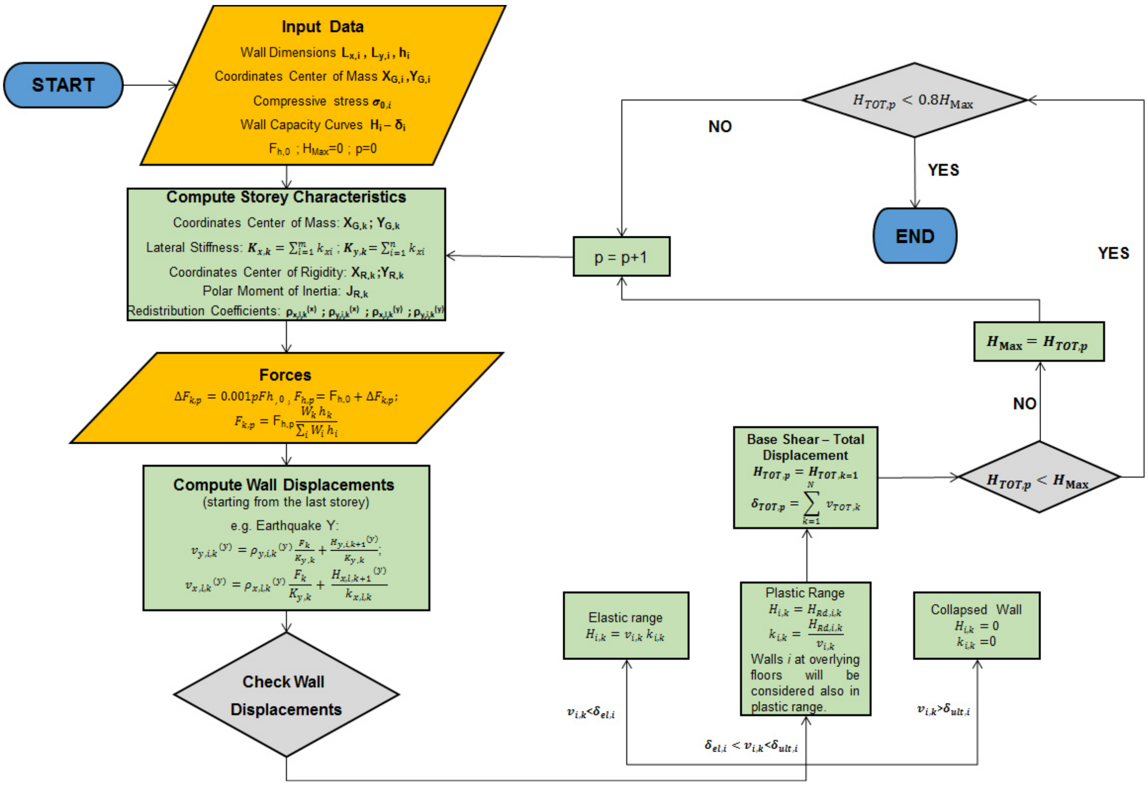

Fig. 2. Flowchart of the E-PUSH algorithm

At the jth step of the iterative procedure, the horizontal force $F_{k,j}$, obtained increasing by ΔF_k the force $F_{k,j-1}$ at the (j-1)th step, is applied in the center of mass of the floor, independently in the x direction and in the y direction and the analysis starts from the top floor of the building, k=N. For example, referring to forces acting in y direction, at the first step of the procedure, when all the walls are in the elastic range, the redistribution coefficients, $\rho_{y,i,k}{}^{(y)}$, for the ith, $1 \le i \le m$, shear wall in y direction, and $\rho_{x,l,k}{}^{(y)}$ for the lth, $1 \le l \le n$, shear wall in y direction, result:

$$\rho_{y,i,k}{}^{(y)} = 1 + \frac{K_{y,k}}{J_{R,k}} e_{x,k} x_i; \quad \rho_{x,l,k}{}^{(y)} = \frac{K_{y,k}}{J_{R,k}} e_{x,k} y_k, \tag{12}$$

being $K_{y,k}$ and $K_{y,k}$ expressed by Eq. (8). The inter-story drift of the walls, $v_{y,k}{}^{(y)}$ and $v_{x,k}{}^{(y)}$ are thus

$$v_{y,i,k}{}^{(y)} = \rho_{y,i,k}{}^{(y)} \frac{F_k}{K_{y,k}}\,; \; v_{x,l,k}{}^{(y)} = \rho_{x,l,k}{}^{(y)} \frac{F_k}{K_{y,k}}. \tag{13}$$

In each step of the procedure, the inter-story drift of each shear wall is compared with the elastic drift and the ultimate drift previously defined. Three different situations can occur:

1) the drift of the wall is lower than δ_e. In this case, the wall is still in its elastic range, the shear force in the wall is

$$H_{y,i,k}{}^{(y)} = v_{y,i,k}{}^{(y)} k_{y,i,k}\,; \; H_{x,l,k}{}^{(y)} = v_{x,l,k}{}^{(y)} k_{x,l,k} \tag{14}$$

and its stiffness of the wall is not modified;

2) the drift of the wall is higher than δ_e and lower than δ_u. In this case, the wall is in the plastic range, and the shear force in the wall is equal to its resistance $H_{Rd,i}$ and its lateral stiffness is reduced to take into account cracks in the wall;

$$k_{y,i,k} = \frac{H_{Rd,i,k}}{v_{y,i,k}{}^{(y)}};\; k_{x,l,k} = \frac{H_{Rd,l,k}}{v_{x,l,k}{}^{(y)}} \tag{15}$$

3) the drift of the wall is higher than the ultimate displacement δ_u. In this case, the wall is collapsed according the bilinear resistance envelope previously defined; the shear resistance of the wall and its stiffness are set to zero and the wall is assumed to sustain only vertical loads.

When the abovementioned checks on all the walls of the kth floor are completed, it is possible to update the corresponding shear resistance of the story

$$H_{y,TOT,k} = \sum_{i=1}^{n} H_{y,i,k}{}^{(y)}; \tag{16}$$

its total stiffness

$$k_{y,TOT,k} = \sum_{i=1}^{n} k_{y,i,k}{}^{(y)}\,; \tag{17}$$

and the drift

$$v_{y,TOT,k} = \frac{H_{y,TOT,k}}{k_{y,TOT,k}}. \tag{18}$$

Once the analysis of the *k*th floor is concluded, it is possible to move to the underlying (*k*-1)th story, repeating the procedure described before. It must be highlighted that the redistribution coefficients for the story force are then computed as in Eq. (12), while the displacement in each wall is evaluated as the sum of: the story displacement redistributed according to the coefficients, $\rho_{y,i,k}{}^{(y)}$ or $\rho_{x,l,k}{}^{(y)}$, and the displacement due to the shear in the overlying wall, $H_{y,i,k+1}{}^{(y)}$ or $H_{x,l,k+1}{}^{(y)}$, previously computed.

$$v_{y,i,k}{}^{(y)} = \rho_{y,i,k}{}^{(y)} \frac{F_k}{K_{y,k}} + \frac{H_{y,i,k+1}{}^{(y)}}{K_{y,k}}; \qquad (19.a)$$

$$v_{x,l,k}{}^{(y)} = \rho_{x,l,k}{}^{(y)} \frac{F_k}{K_{y,k}} + \frac{H_{x,l,k+1}{}^{(y)}}{k_{x,l,k}}. \qquad (19.b)$$

The procedure is then repeated for the remaining underlying story in order to complete the analysis of the building for the acting forces, assumed distributed, for example, according to Eq. (11), so that the base shear resistance $H'_{y,TOT,1}$ and the total displacement at *N*th floor, $\delta'_{y,TOT}$, are obtained at the end.

At this stage, forces defined in eq. (11) are incremented by ΔF_k, as already said, and the procedure is repeated floor by floor in descending order, duly updating the stiffness and the center of stiffness of each floor.

It must be emphasized at this point a key peculiarity of the method: when the inter-story drift of a wall is in the plastic range, i.e. bigger than δ_e, at the *k*th floor, in the subsequent steps of the algorithm the overlying walls, $k<j\leq N$, are also assumed in the plastic range, and therefore not able to sustain an increase of the shear forces, while underlying walls are not directly affected. This aspect is duly taken into account when center of stiffness and stiffness are updated.

The algorithm is run incrementing the forces until the base shear resistance $H'_{y,TOT,1}$ reduces to about 80% of the relative maximum base shear resistance, defining in this way the capacity curve of the whole structure.

4. CAPACITY DEMAND ASSESSMENT

Once determined the capacity curve of the building as presented in the previous paragraph, the seismic demand imposed on the building is derived according the procedure presented in ATC-40 [14] and FEMA-274 [15] with specific reference to the Italian Building Code [12] and to the N2 method developed by the University of Ljubljana [16]. The procedure has been implemented according the following steps:

1) transformation of the non-linear capacity curve in an equivalent bilinear curve, as illustrated in Fig. 3, being the maximum force $F_{y,eq}$ the mean between the maximum base shear in the non-linear capacity curve and the base shear corresponding to the attainment of the elastic limit of the capacity curve; the yield displacement $\delta_{y,eq.}$ as the ratio between $F_{y,eq}$ and the effective stiffness of the structure K_E, and the ultimate displacement $\delta_{u,eq.}$ as the maximum displacement in the capacity curve.
2) conversion of the bilinear, force-displacement capacity curve, $F - \delta$, of the multi-degree of freedom (MDOF) system, in an acceleration-displacement, $S_a - S_d$, capacity diagram, for an equivalent single degree of freedom (SDOF) system, according the following formulae

$$S_a = \frac{F}{\Gamma m^*};\ S_d = \frac{\delta}{\Gamma};\ \ m^* = \sum_{j=1}^{N} m_j \phi_j \qquad (20)$$

where Γ is the mass participation factor

$$\Gamma = \frac{\sum_{j=1}^{N} m_j \phi_j}{\sum_{j=1}^{N} m_j {\phi_j}^2} \qquad (21)$$

m^* is the equivalent mass of the equivalent SDOF system, m_j are the story masses and ϕ_j are the normalized displacements in the considered direction, so that the elastic period of the idealized bilinear system is given by

$$T^* = 2\pi \sqrt{\frac{m^* S_{d,y}}{S_{a,y}}};. \qquad (22)$$

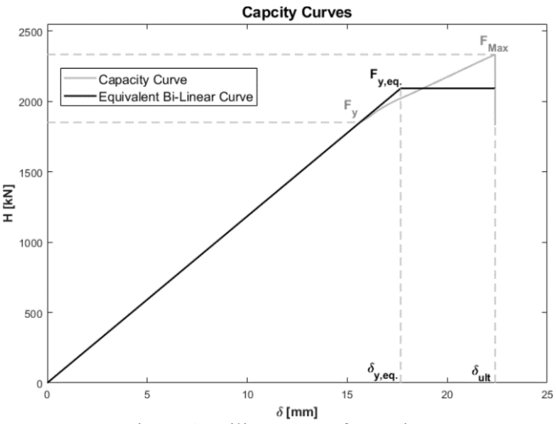

Figure 3. Bilinear transformation

3) Conversion of the elastic design spectrum defined in the Italian Building code from the standard pseudo acceleration-natural period, $S_{ae} - T$, to the pseudo acceleration-displacement format $S_{ae} - S_{de}$, in order to obtain the demand diagram,

$$T^* = 2\pi \sqrt{\frac{m^* S_{d,y}}{S_{a,y}}};. \qquad (22)$$

being.

$$S_{de} = \frac{T^2}{4\pi^2} S_{ae};. \qquad (23)$$

4) plot of capacity spectrum and demand spectrum curves in the same graph, to define displacement demand. It may happen that capacity curve intersects demand curve: in this case the displacement demand is assumed equal to the intersection point d_t, otherwise, the displacement demand is determined according the N2 method [16], summarized in the following;
5) evaluation of the seismic performance of the structure comparing the displacement demand d_t with the ultimate displacement d_u defined by the capacity curve.

In the N2 method, starting from the intersection of the radial line corresponding to the elastic period T^* with the elastic design spectrum defining the acceleration demand $S_{ae}(T^*)$ and the corresponding elastic displacement demand $S_{de}(T^*)$, the reduction factor R_μ can be obtained as ratio between the acceleration demand $S_{ae}(T^*)$ and the yield acceleration $S_{a,y}$

$$R_\mu = \frac{S_{ae}(T^*)}{S_{a,y}}. \qquad (24)$$

If the elastic period T^* is larger than or equal to the characteristic period of the ground motion T_C, the ductility demand μ, defined as the ratio between the displacement demand and the yield displacement ($\mu = d_t / S_{d,y}$), is equal to R_μ, therefore

$$d_t = S_{de}(T^*);\ \mu = R_\mu \quad \text{for } T^* \geq T_C \qquad (25)$$

If the elastic period $T^* < T_C$, the ductility demand μ and the displacement demand d_t can be calculated by

$$\mu = (R_\mu - 1)\frac{T_C}{T^*} + 1;\ d_t = \frac{S_{de}(T^*)}{R_\mu}\left(1 + (R_\mu - 1)\frac{T_C}{T^*}\right). \quad (26)$$

Since the final aim of the assessment is the evaluation of the seismic risk index I_R of the structure, which is the ratio between the peak ground acceleration associated to the structural capacity PGA_C and the peak ground acceleration associated to the design spectrum PGA_D

$$I_R = \frac{PGA_C}{PGA_D}, \quad (24)$$

and the design spectrum and the PGA are defined in terms of reference return period T_R, the procedure for the definition of the displacement demand d_t is iterated according different return periods until the displacement demand is equal to the ultimate displacement d_u defined by the capacity curve. In this way, the reference return period corresponding to the capacity of the structure $T_{R,C}$ is evaluated together with the associated peak ground acceleration PGA_C.

In Fig. 4, an example is reported for the verification in the acceleration-displacement response spectra (ADRS) plane. In the Figure, they are represented: the bilinear capacity curve of the structure (red solid line); the design spectrum (blue solid line) for a return period of 712 years, referring to the ultimate limit state for life safety of occupants, and the corresponding inelastic spectrum (blue dashed line); the displacement demand (scarlet dashed line) to be compared with the ultimate displacement (green dashed line).

The elastic spectrum (green solid line) for the return period consistent with the capacity of the structure $T_{R,C}$ and the corresponding inelastic spectrum (green dashed line)

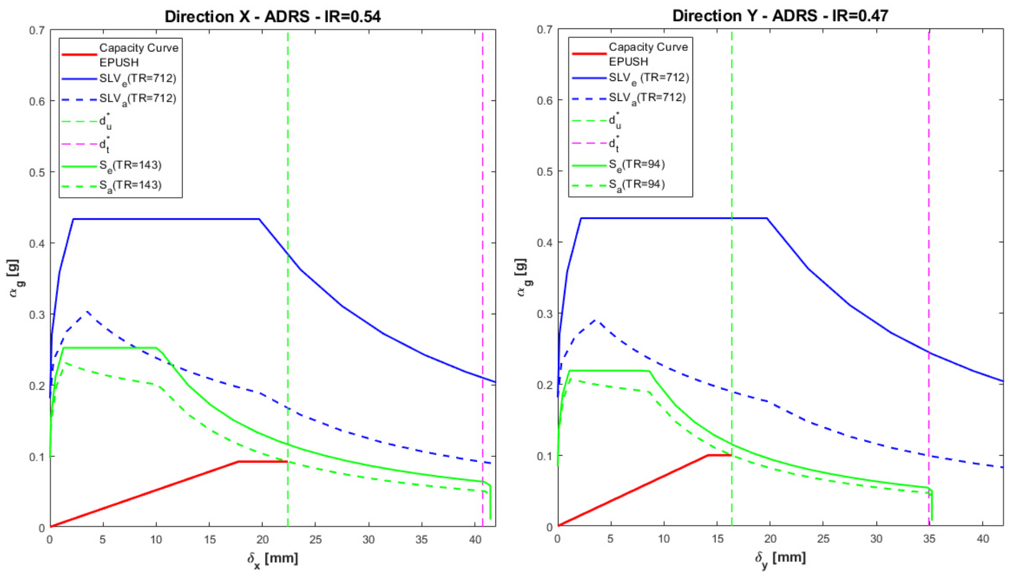

Figure 4. Verification in the ADRS plane

are obtained via E-PUSH algorithm and capacity curves in red are obtained with Aedes.

5. VALIDATION OF THE METHOD

The proposed method for non-linear static analysis of masonry buildings has been validated compared the outcomes of the algorithm with those obtained by means of a commercial pushover analysis software called Aedes PCM [17] considering several masonry buildings. The results agree very satisfactorily both in terms of ultimate resistance, as well as in terms of ultimate displacement, in all considered case studies, so validating the program. A deep discussion of this topic is out of the scope of the present paper, therefore only a short illustration is reported here.

The comparison refers to a two-story building located in Florence, whose E-PUSH model was already shown in Fig. 1.

In Fig. 5, they are compared the 3D model of the building built for E-PUSH algorithm (on the left side) and the 3D equivalent frame model used in Aedes.

The results, obtained considering drift or ductility limits, are compared in diagrams in Fig. 6, where capacity curves in green are obtained via E-PUSH algorithm and capacity curves in red are obtained with Aedes.

6. CONCLUSIONS

An innovative programme for non-linear static analysis of masonry buildings has been illustrated.

The programme, which is a pushover-type algorithm, is based on very general assumptions and take into account shear walls extended till to foundation, allows to arrive to very simple and effective 3D model, avoiding the inconsistencies, which are typical of the classic pushover programmes, based in 3d frame equivalent approach.

The programme, which is very quick and does not require particularly skilled users, has been validated comparing the outcomes obtained analyzing several masonry buildings with those obtained using commercial software,

The results confirm the reliability and the effectiveness of the program, so justifying further studies and improvements.

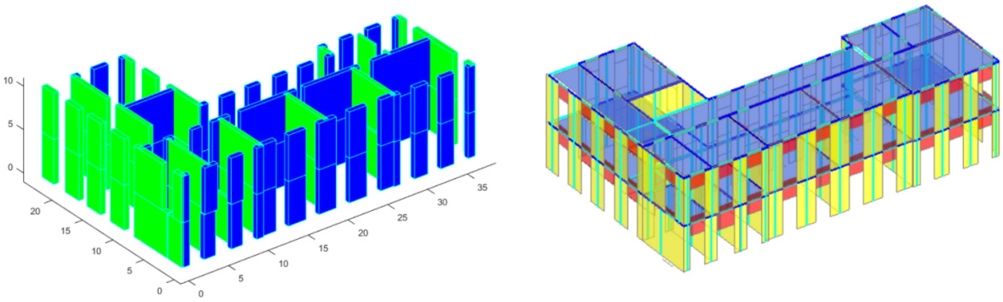

Fig. 5. 3D- Layout of the resistant shear walls (E-PUSH) and equivalent frame model in Aedes PCM

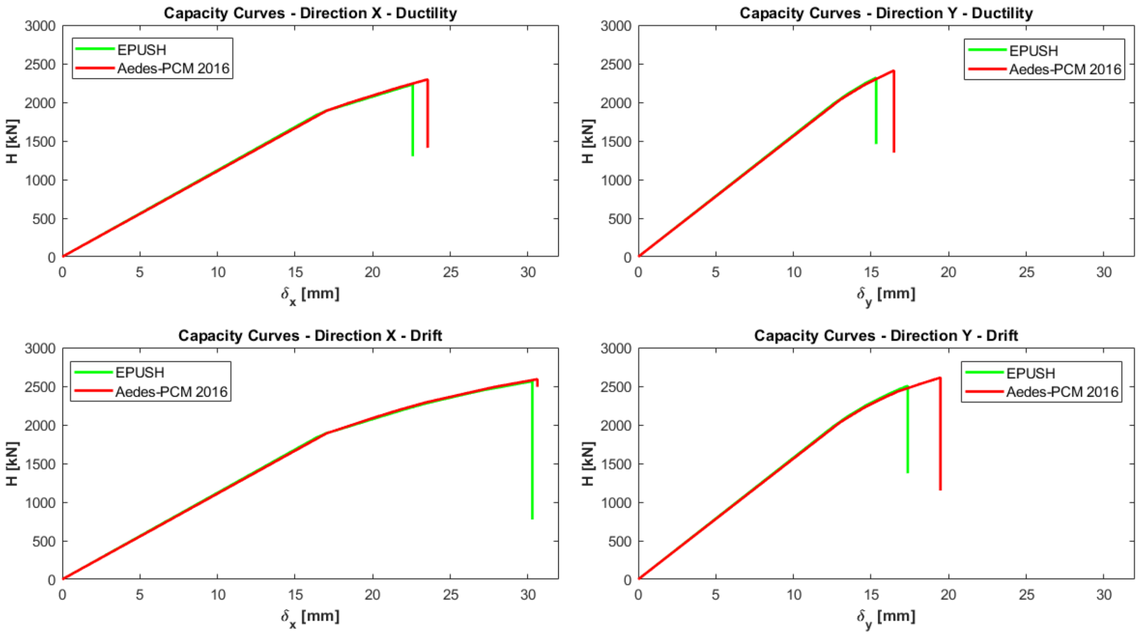

Fig. 6. Comparison of capacity curves obtained via Aedes PCM (in red) and E-PUSH (in green)

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