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ON THE USE OF PUSHOVER ANALYSIS FOR EXISTING MASONRY BUILDINGS

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SUMMARY

The application of nonlinear static (pushover) procedures for the assessment of existing masonry buildings has been introduced into seismic codes (e.g. EC8, new Italian Seismic Code OPCM 3274/03), but it still includes several critical points in the implementation to real structures.
The three-dimensional model of a masonry building can be obtained by assembling frame-type macro-element models of the walls and orthotropic membrane elements in order to represent the mechanical behaviour of flexible floors. This modelling, although very effective in representing the actual behaviour, does not allow to use common simplifications such as rigid floor motion. Moreover, a 3D pushover algorithm requires a predefined pattern of horizontal forces to be applied to the structure and, keeping constant the relative force ratios, the horizontal displacement of a control node is incremented. A new displacement-based algorithm for the adaptive pushover analysis of masonry walls and buildings is presented: the load pattern, in this case, is directly derived, step-by-step, by the actual deformed shape evaluated during the pushover analysis. The proposed procedure seems to be very powerful for in-plane analyses of walls, whilst it requires some corrections in order to be applied to three-dimensional masonry buildings.

1. INTRODUCTION

The recent trend in seismic codes is oriented to a simplified mechanical approach in order to assess buildings seismic performances using procedures which are based on non-linear static analyses and on the Capacity Spectrum Method [Freeman S., 1998]. This method considers the non-linear behaviour of structures by means of their capacity curve, which can be obtained reducing the pushover analysis result through the definition of a "substitute" [Shibata and Sozen, 1976] s.d.o.f. equivalent system. The seismic demand can be then estimated, in terms of spectral displacement (performance point), intersecting the so called Capacity Spectrum (the Capacity Curve plotted in terms of spectral acceleration and displacement) with the earthquake response spectrum, plotted in AD format (acceleration vs. displacement) and properly reduced to take into account the effects of energy dissipation related to non-linear structural response. This non-linear static approach aims at predicting the maximum horizontal displacement resulting from a dynamic analysis.
Among the different methods developed in the last years, this work will follow the inelastic (constant ductility) response spectrum method described by Fajfar [Fajfar, 2000]. The application of this method to European building typologies points out the difficulties related to existing masonry buildings; the methodology, developed for concrete structures, shows some problems when applied to URM structures in relation to the own peculiarities of this structural system. Traditional masonry buildings have often very flexible diaphragms (usually made of wood) and this prevents from assumption of rigid floors. Furthermore a different architectural disposition of walls may contribute to localize damage to 'unexpected' parts of building, due to irregularities in elevation. In this work the peculiarities of the global response of masonry buildings are shown with the aid of a

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simplified non-linear element model, able to reproduce earthquake damage to masonry buildings and failure modes observed in experimental tests.

2. NUMERICAL MODEL

The 3-dimensional modelling of whole URM buildings starts from some hypotheses on their structural and seismic behaviour: 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), on which the horizontal actions distribution between the walls depends; the local flexural behaviour 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 behaviour (a global seismic response is possible only if vertical and horizontal elements are properly connected). The wall is modelled as a frame of non-linear element, which constitutive relationship is formulated to approximate the actual damage behaviour of masonry panels. The numerical models and analysis procedures, described in the rest, have been incorporated into the TREMURI program [Galasco et al., 2002].

2.1. Model of the wall in-plane behaviour

A frame-type representation of the in-plane behaviour of masonry walls is adopted: each wall of the building is subdivided into piers and lintels (modelled by non-linear beams) connected by rigid areas (nodes). Earthquake damage observation shows, in fact, that only rarely (very irregular geometry or very small openings) cracks appear in these areas of the wall: because of this, 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 stringcourses (beam elements), tie-rods (non-compressive rod elements), previous damage, heterogeneous masonry portions, gaps and irregularities can be easily included in the structural model.
The non-linear macro-element model, representative of a whole masonry panel, is adopted for the 2-nodes elements representing piers and lintels. Rigid end offsets are used to transfer static and kinematic variables between element ends and nodes.

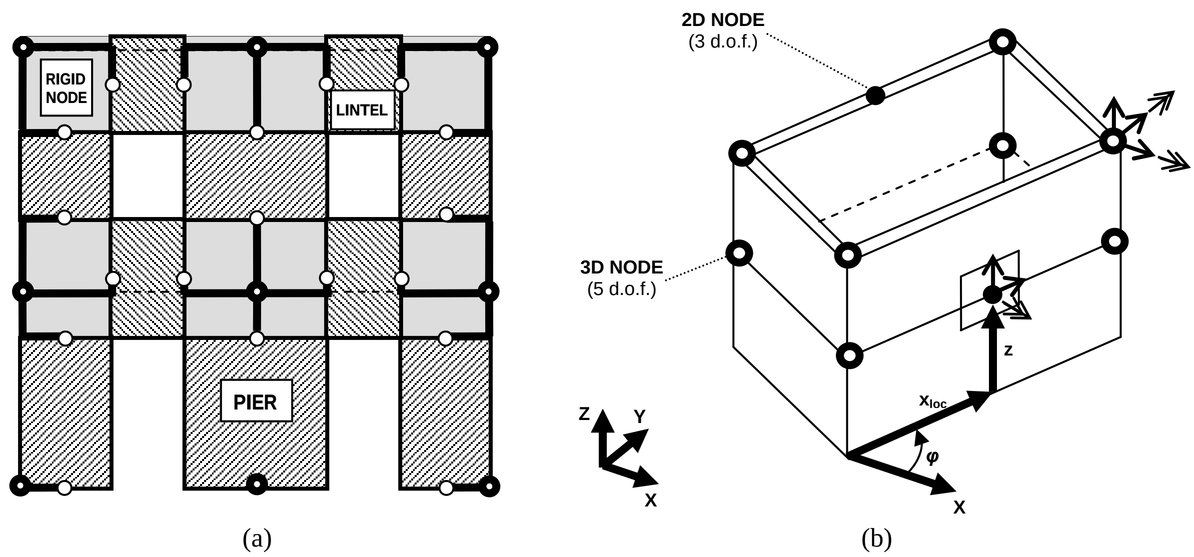


Figure 1: Macro-element modelling of a masonry wall (a); 3-D building model assembling (b).

2.2. Three-dimensional model

A global Cartesian coordinate system (X,Y,Z) is defined and the wall vertical planes are identified by the coordinates of one point and the angle formed with X axis. In this way, the walls can be modelled as planar frames in the local coordinate system and internal nodes can still be 2-dimensional nodes with 3 d.o.f..
The 3D nodes connecting different walls in corners and intersections need to have 5 d.o.f. in the global coordinate system (u_X, u_Y, u_Z, rot_X, rot_Y): the rotational degree of freedom around vertical Z axis can be neglected

because of the membrane behaviour adopted for walls and floors. These nodes can be obtained assembling 2D rigid nodes acting in each wall plane (Figure 1.b) and projecting the local degrees of freedom along global axes.
Floor elements, modelled as orthotropic membrane finite elements, with 3 or 4 nodes, are identified by a principal direction, with Young modulus E_1, while E_2 is the Young modulus along the perpendicular direction, ν is the Poisson ratio and $G_{1,2}$ the shear modulus: E_1 and E_2 represent the wall connection degree due to the floors, by means also of stringcourses and tie-rods. $G_{1,2}$ represents the in-plane floor shear stiffness which governs the horizontal actions repartition between different walls.
Since the 2D nodes do not have degrees of freedom along the direction orthogonal to the wall plane, in the calculation, the nodal mass component related to out-of-plane degrees of freedom is shared to the corresponding dofs of the nearest 3D nodes of the same wall and floor. This solution then allows to carry out static and dynamic analyses with components of the seismic acceleration applied along any direction.

2.3. Non-linear beam element

A non-linear beam element model has been implemented in the TREMURI program [Galasco et al. 2002] in together with the macro-element with additional degrees of freedom, described by:

1) initial stiffness given by elastic (cracked) properties;
2) bilinear behaviour with maximum values of shear and bending moment as calculated in ultimate limit states;
3) redistribution of the internal forces according to the element equilibrium;
4) detection of damage limit states considering global and local damage parameters;
5) stiffness degradation in plastic range;
6) secant stiffness unloading;
7) ductility control by definition of maximum drift (δu) based on the failure mechanism, according to the Italian seismic code:

$$\delta_u = \frac{(u_j - u_i)}{h} + \frac{(\varphi_j + \varphi_i)}{2} = \begin{cases} 0.4\% & \text{shear} \\ 0.6\% & \text{bending} \end{cases} \qquad (1)$$

8) element expiration at ultimate drift without interruption of global analysis.

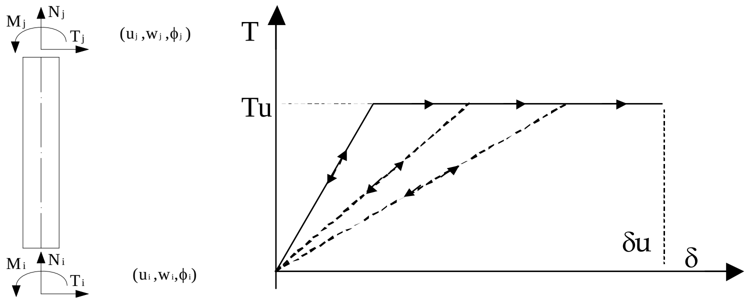
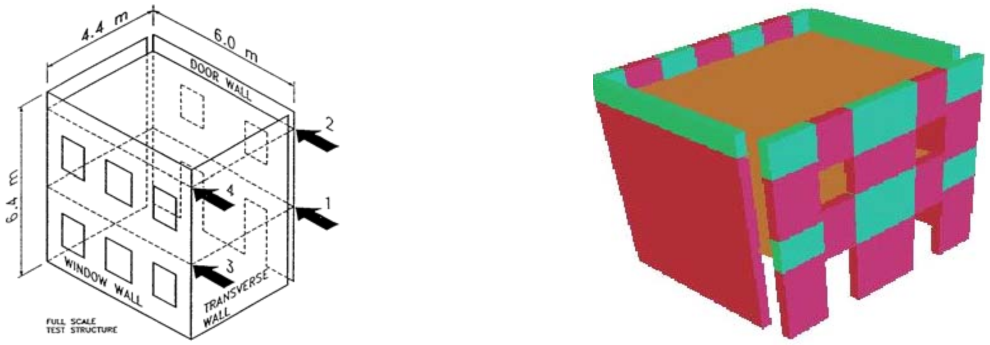


Figure 2: Non-linear beam degrading behaviour.

2.4. Numerical simulation of experimental testing

In order to demonstrate the reliability of this simplified model, a numerical simulation of experimental testing on a full scale masonry building carried out in the Laboratory of the University of Pavia [Magenes et al., 1995] is here presented.



(a) (b)

Figure 3: Scheme of the test (a); 3-dimensional view of the macro-element model (b).

As shown in Figure 3, the experimental tests have been carried out on two separated structural systems (the isolated “door” wall and the “window” wall connected to the two transverse walls) under cyclic loads obtained requiring the same force on each floor and each level. The numerical simulation has then performed by a monotonic analysis of two different plane models with the same mechanical characteristics to reproduce the result of the tests. A cyclic analysis was not realistic because of the simplified unloading behaviour of the non linear beam used, but the monotonic pushover result can be anyway compared with the maximum envelope of test cycles. Numerical and experimental results are in good agreement, both in terms of base shear-second floor displacement curves (Figure 4) and damage localization at the different load steps.
An experimental evaluation of drift (0.8% for shear and 1% for bending) has been then performed since the values suggested by the code were excessively conservative.

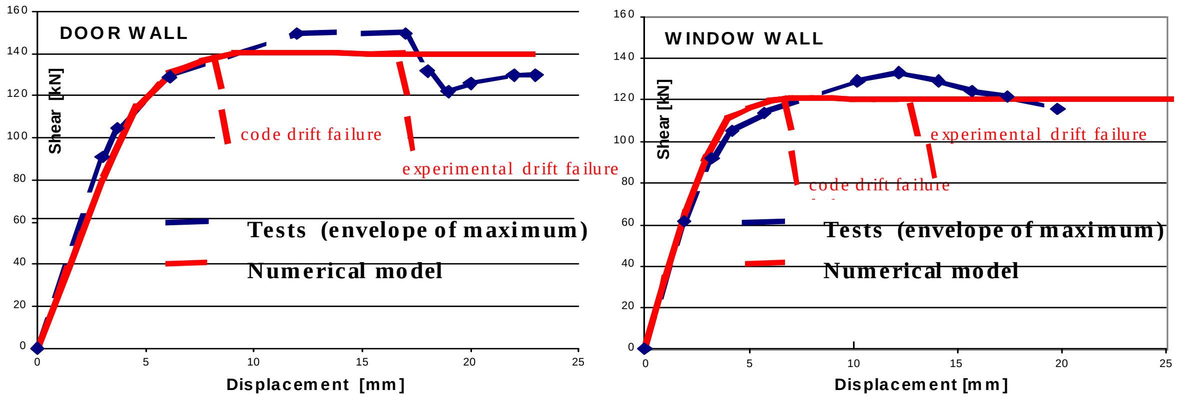


Figure 4: Comparison between experimental envelopes and numerical simulation

3. NONLINEAR STATIC ANALYSIS PROCEDURE

The non-linear static analysis procedure adopted in the EC8 and in the Italian seismic code, both for design and assessment, is based on a maximum displacement prediction, which depends on the definition of an equivalent elastic perfectly plastic s.d.o.f. structure, derived from a capacity curve obtained by a pushover analysis. The mathematical formalization and the assumed simplifications have been adapted to masonry structures, but a close examination is required especially for some specific steps of the pushover algorithm. This kind of analysis requires a predefined pattern of horizontal forces to be applied to the structure and, keeping constant the relative force ratios, the horizontal displacement of a control node is incremented. However the choice of control node and force distribution is not univocal and results may depend on it.

3.1. Control node choice

The pushover analysis requires the selection of a control node, in order to apply the analysis procedure, using the algorithm described by Galasco et al. [2004] and developed by matrix operations based on a master d.o.f.. The selected master d.o.f. displacement may be different from the d.o.f. displacement plotted at the end in the capacity curve, usually chosen among the top level nodes.

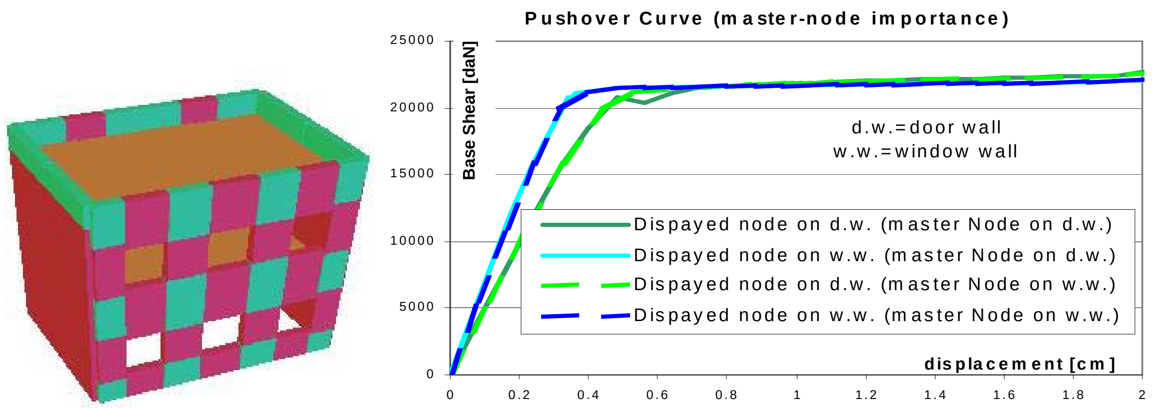


Figure 5: Capacity curves for different selections of master-control nodes in the prototype model.

3.1.1. Master node importance

The first issue to be investigated is the dependency of pushover curves on master d.o.f. selection. Resuming the prototype tested in Pavia, a complete 3D numerical model has been carried out. The two main walls ("window" and "door") have different stiffness and different ultimate strength, so the choice of the position of the master node has to be investigated in case of same control node displayed.

The displayed node displacements are very similar independently from the chosen master node. However the curves may be significantly different close to the failure condition: the position of the master node on the weaker wall provides an exhaustive description of collapse, while with the position of the master node on the stronger wall, only the incipient failure step of analysis can be obtained, since the following analysis step cannot be performed due to the collapse of the weaker wall.

By observing the capacity curves plotted in Figure 6, it can be noticed that only choosing the master node on weak wall, the actual collapse behaviour is evident. As the post failure behaviour of the building is not useful for this work, the choice of the master node is not a major problem; even if it is preferable to put it on the weakest wall, since different choices allow in any case to catch the building resistance and its failure, no further remarks on this issue will be discussed.

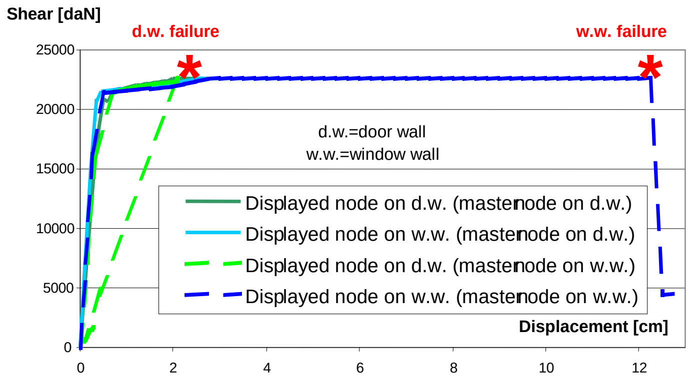


Figure 6: Master node importance: walls failure

3.1.2. Control node dependency

Looking at the previous curves, a difference between weak or strong wall displacements is evident: the window wall is weaker but more ductile and, in terms of capacity spectrum method, it shows more displacement capacity compared to the strong one. The contradiction is evident: the same building seems to offer a different resistance depending on the chosen control node.

The conversion to the s.d.o.f. equivalent structure is obtained by dividing both shear and displacement by the participation factor Γ,

$$\Gamma = \frac{\sum m_i \Phi_i}{\sum m_i \Phi_i^2}, \qquad (2)$$

where, for each node, m_i is the nodal mass and $\mathbf{\Phi}_i$ is the first mode displacement scaled so that it is equal to 1 for the control d.o.f.. Due to the elastic definition of Γ, the initial branch of the curves is similar, while the last points, close to collapse, can be different.

This problem concerns mainly masonry buildings, where a storey centre of mass can not be assumed as a control node. However an average (weighted on nodal masses) floor displacement allows a generalized description of the structure behaviour, independent from the different stiffnesses and strengths of the walls, identifying a single failure point. This approach is followed in the rest of the paper.

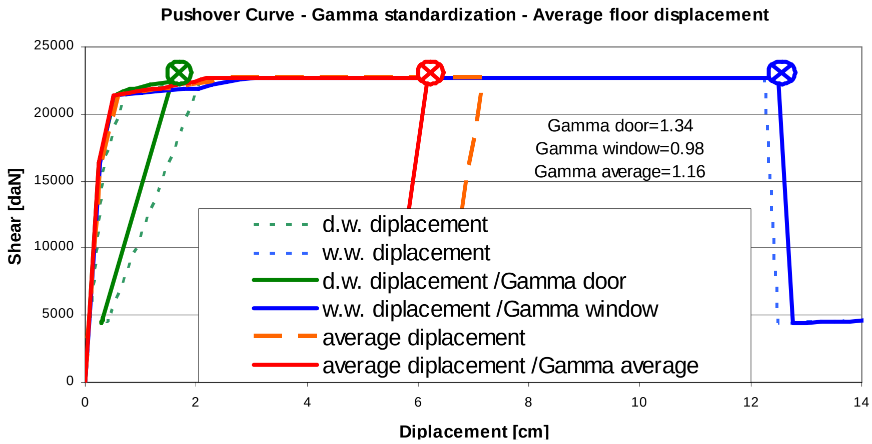


Figure 7: Comparison of s.d.o.f. pushover curves.

3.2. Force distribution

The basic assumption of non-linear static analysis is that the pushover curve is an envelope of the responses of dynamic analyses; the accuracy of this assumption is strongly dependent on a correct choice of the initial force distribution. Two possible distributions are commonly adopted: modal and uniform.

The first one is able to represent the structural dynamic amplification, which increases the action on higher storeys; on the contrary, the second one can describe the behaviour of a building under extensive damage, preventing force redistributions among levels (in particular the action on lower levels cannot shift to higher ones). These two distributions may be assumed as boundary conditions for seismic analyses: the actual result, coming from dynamic analyses, is assumed to be within these two solutions and the real failure mode is predicted by one of the two distributions.

In the following a case study showing two different failure mechanisms related to the two different force distributions is presented. The structure is a three-storey brick masonry building with flexible diaphragms. The plan and front views are reported in Figure 8.

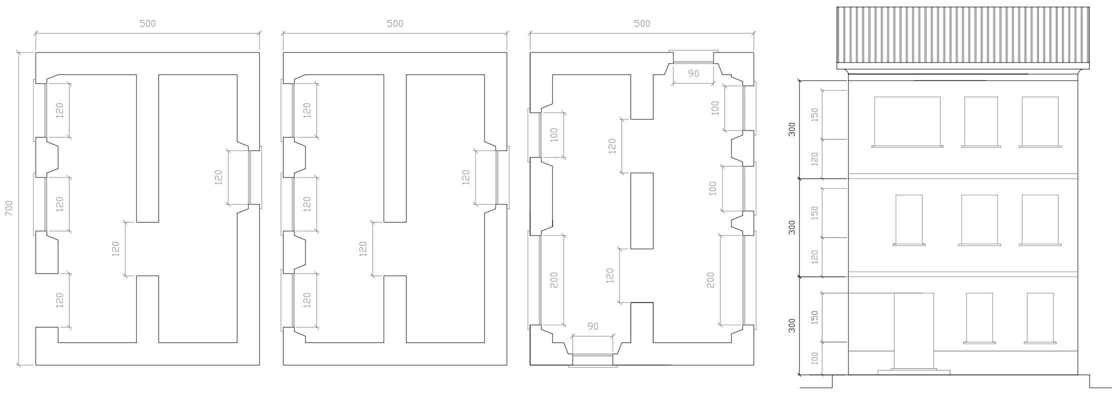


Figure 8: Case study building drawings (measures quoted in cm)

Three pushover curves, starting from different initial force distributions, are shown: one (a) calculated from uniform distribution, the others from modal distribution according to the exact modal forces (b) and according to the triangular approximation (c) (Figure 9).

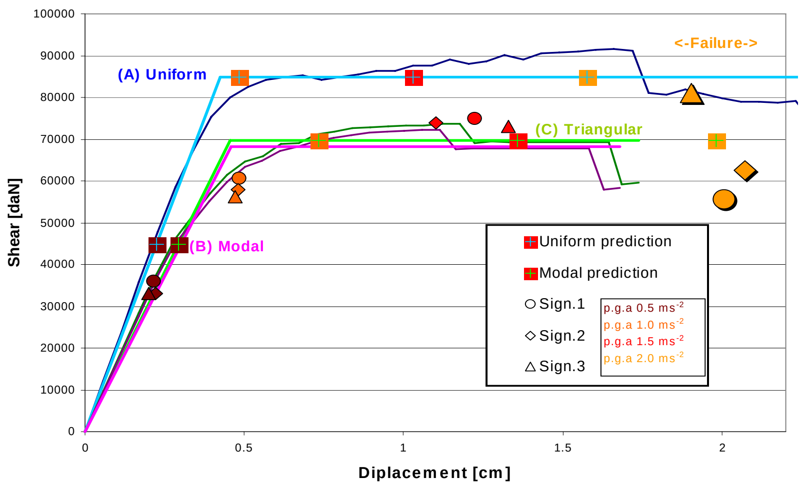


Figure 9: Comparison of pushover curves and dynamic response (signals 1, 2, 3).

Each curve is associated to its bilinear simplification (equal energy criterion) using the s.d.o.f. elastic perfectly plastic approximation. The pushover analyses show a different failure mechanism: uniform distribution causes a first floor soft storey mechanism, while in the modal ones induce distributed damage in the upper levels. In order to test the accuracy of the method and evaluate the actual collapse mechanism, several dynamic analyses have been carried out with different strong motions (three synthetic signals numbered 1, 2, 3 compatible with soil type B of EC8) for increasing PGA values. The results allow to evaluate the reliability of the prediction as formulated by Fajfar method [Fajfar, 2000]: the maximum displacement demand is calculated from the pseudo-acceleration spectrum, reduced according to the ductile capacity derived from the non-linear static analysis.

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Table 1. Comparison between static predictions and dynamic non linear analyses

	Non linear static		Non linear dynamic		
PGA [ms^{-2}]	Unif. [cm]	Modal [cm]	Sign.1 [cm]	Sign.2 [cm]	Sign.3 [cm]
0.5	0.22	0.29	0.21	0.20	0.22
1.0	0.48	0.73	0.48	0.47	0.48
1.5	1.03	1.36	1.22	1.32	1.15
2.00	1.58	1.98	(2.01)	(1.91)	(2.10)

Higher levels of PGA cause collapse of the building and the maximum displacement values are not reliable (they have been put into brackets). From the observation of the deformation and the damage distribution of dynamic analyses, a top level collapse has been noticed, according to modal distribution pushover analysis.
The collapse mechanism mismatch between the initial fixed distributions may require to perform some non linear dynamic analyses in order to catch the real failure mode, as the two distributions define two boundary responses for the structure under seismic action. Although, from a design code point of view, the worst situation may be assumed to guarantee the structural safety, from a research point of view a consistent method is required and hence a new pushover approach is presented in the following.

4. ADAPTIVE PUSHOVER PROCEDURE BASED ON ACTUAL DISPLACEMENTS

As highlighted before, the initial force distributions try to match the dynamic effects, but the choice of fixed values may be a limitation: the structure indeed, as the damage progresses, changes its response from the original dynamic amplification, well represented by a modal force distribution, to a near collapse condition, better described by a uniform force distribution.
Several versions of pushover analysis have been suggested by some authors [Chopra and Goel, 2002; Antoniou and Pinho, 2004] in order to consider the progressive building damage by evolutive change of force ratios. In the force-based adaptive pushover approach, a modal analysis is performed step by step to update the force modal ratios; in the dispacement-based adaptive pushover, the modal shape is directly imposed to the structure, using a displacement control analysis.
In this paper, a different approach is suggested: the previous methods require the damage stiffness matrix definition in order to solve the modal problem in a different damaged state while, actually, the non-linear analyses, performed by the TREMURI program, adopt a Newton-Raphson procedure without any degraded

matrix definition. Instead of multimodal shape, the actual deformed shape, evaluated during the pushover analysis at the previous step, can be used to set the force ratios at the current step.
The adaptive procedure can be described as:

$$\begin{aligned}
&\{\boldsymbol{f_0}\} = p_0[\mathbf{M}]\{\psi_0\} && \text{starting step}\\
&\{\boldsymbol{f_1}\} = p_1[\mathbf{M}]\{X_0\} && \text{step 1}\\
&\{\boldsymbol{f_2}\} = p_2[\mathbf{M}]\{X_1\} && \text{step 2} \qquad (3)\\
&\ldots\ldots\ldots\ldots\ldots\ldots && \ldots\ldots\\
&\{\boldsymbol{f_i}\} = pi[\mathbf{M}]\{X_{i-1}\} && \text{step i}
\end{aligned}$$

where for the generic i-step $\{\boldsymbol{f_i}\}$ is the force ratio, p_i is the partecipation coefficient (irrelevant in this case since relative ratios are applied), $[\mathbf{M}]$ is the mass matrix , $\{\boldsymbol{\psi_0}\}$ is the initial first mode eigenvector and $\{\mathbf{X}_i\}$ is the current shape.
This actual displacement based pushover (ADAP) procedure was firstly applied to the in-plane analysis of the façade of a masonry building (Giuncugnano Hall, see details in Galasco et al., 2005).
A good agreement has been found between the response of ADAP procedure and dynamic analysis results, as shown in Figure 10..

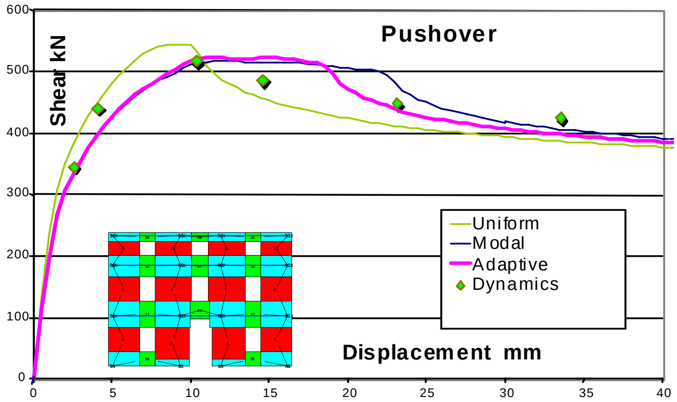

Figure 10: Pushover curves of the Giucucnano Hall façade.

Furthermore the initial force distribution used in the algorithm is not really important: indeed the curves converge after few steps to the same result even if the initial force ratio is different. This happens as the actual shape approaches the modal damage shape.
However the characteristics of masonry buildings highlight the contradictions of this procedure, which are less evident in concrete structure. For an existing structure with wooden floors (see plans in Figure 11) the presence of flexible diaphragms does not allow redistribution between walls; on the contrary, the adaptive algorithm causes redistribution of forces on the weakest wall even if this is not physically consistent.

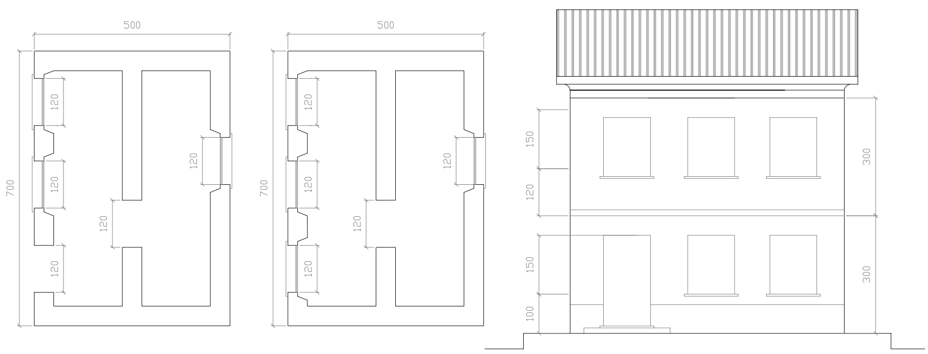

Figure 11: Wooden floor masonry building

The abnormal deformation of the central wall is evident in the plan deformed shape (Figure 12), as well as the excessive weakness of the adaptive curve compared to fixed force pattern pushover curves. This issue was not evident in other building typologies, because of the presence of rigid floors, which govern force redistribution among walls.

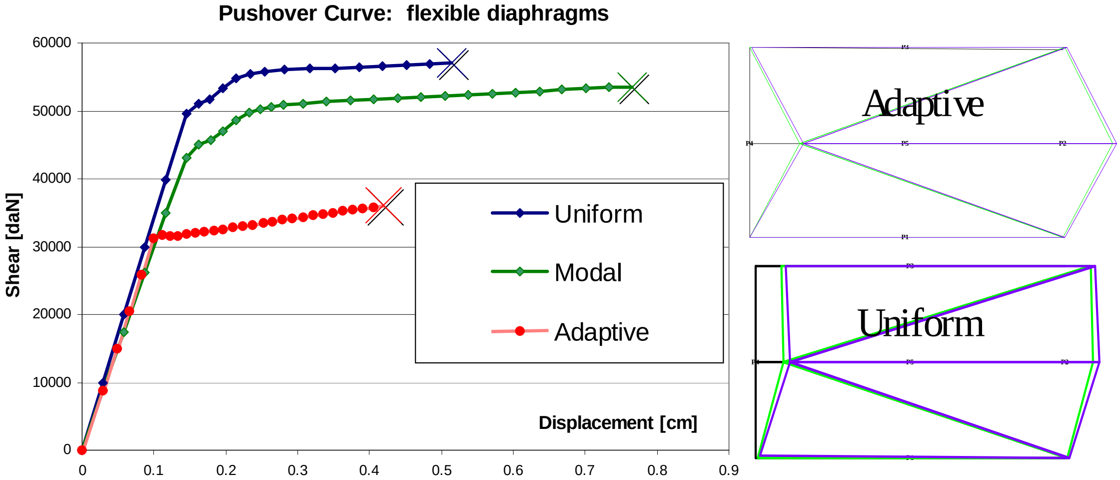

Figure 12: Wooden floor masonry building: pushover curves and plan deformation

Another problem of this approach regards the collapse of upper levels: according to dynamic amplification, greater forces may occur on these levels but the force increment should not exceed the first mode force value (i.e. the modal force distribution value); on the contrary, according to the ADAP algorithm, a further increase occurs. This is not consistent with a dynamic seismic analysis, as shown in Figure 13, for the 3-storey building model .

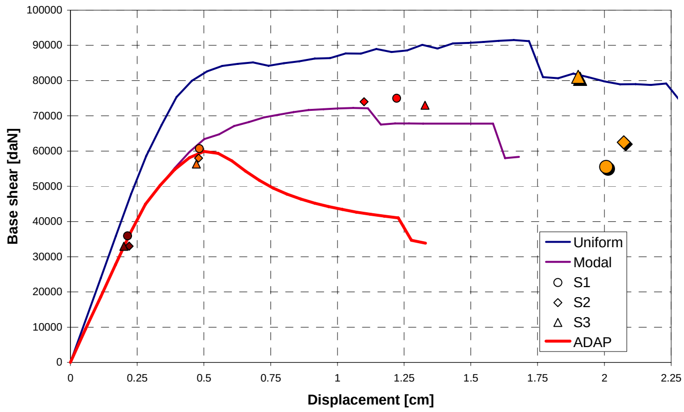

Figure 13: Capacity curve comparison

In this paper, a realistic correction of this procedure is proposed: according to the pushover curve property of being an envelope of dynamic response and according to the boundary meaning of the two fixed distribution of forces (modal and uniform), the force ratios, computed in the current step, can not exceed the boundary distributions. Hence the force distribution is normalized and then compared with the normalized boundary distributions. If one or more components exceed the boundary values, then the component value is set to the normalized boundary one. This iterative process continues until every component is included.
The boundary distribution $\mathbf{f}^{\mathbf{M}}$ is obtained imposing the maximum values of the normalized modal and uniform force distributions, then:

$$\mathbf{f}=\begin{Bmatrix} f_1 \\ f_2 \\ \dots \\ f_n \end{Bmatrix}, \sum_{i=1}^{n} f_i = 1,\ \mathbf{f}\ \text{current distr.} \quad \Rightarrow \quad \mathbf{f}'=\begin{Bmatrix} f'_1 \\ f'_2 \\ \dots \\ f'_n \end{Bmatrix}, \begin{matrix} f'_{i(i\le m)} = f_i^{\mathrm{M}} \\ f'_{i(i>m)} = f_i + \Delta \cdot \dfrac{f_i}{\sum_{i=m}^{n} f_i} \end{matrix} \qquad (1)$$

$$\text{assuming } f_k > f_k^{\mathrm{M}} \text{ for } k \le m \Rightarrow \Delta = \sum_{k=1}^{m} f_k - f_k^{\mathrm{M}} \qquad \sum_{i=1}^{n} f'_i = 1,\ \mathbf{f}'\ \text{"seismic" current distribution}$$

This corrected procedure, consistent with earthquake effects and called seismic adaptive pushover (SDAP), can solve the deficiencies of the ADAP algorithm, as shown in Figure 14, for the cases of flexible floors and top storey failure, previously presented as critical examples.

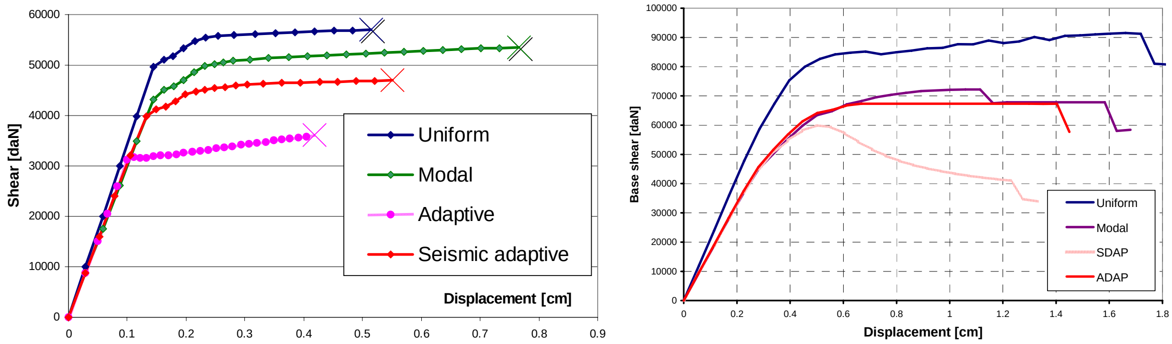

Figure 14: SDAP, ADAP and traditional capacity curves for the 2 case studies

5. CONCLUSIONS

The application of non linear static procedures to existing masonry buildings needs to solve several modelling and analysis problems, which are generally less relevant for other building typologies. Some of these problems have been pointed out in the paper, giving also possible solutions derived from specific studies, by means of several structural analyses, performed with a dedicated analysis program and specifically developed features.
Some of the commonly accepted hypotheses, such as the rigid floor assumption, and their consequences are not valid in this case: the availability of a powerful and versatile tool allowed to remove them and face the analysis problem in a more realistic way. Some effective suggestions have been given in the paper and some case studies are presented.
The developed algorithm for adaptive pushover needs further testing and applications but it seems to be a useful way to better represent the actual dynamic behaviour and to reduce the number of required static analyses: the SDAP algorithm is stable and the pushover analyses performed with TREMURI are rather fast.

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