Abstract

Society’s social and cultural awareness for the importance of historic buildings is leading to a generalized effort for preservation. Most ancient buildings desperately need retrofit in order to mitigate the effects of material aging, of intensive use and of the pathologies resultant from natural disasters. The systemization of seismic retrofitting is the expected course of action.

In Portugal, the implementation of the Eurocodes represents the first step towards the regulation of seismic retrofit of ancient wood and masonry buildings and it is being accompanied by the recovery of traditional constructive techniques, as this Master’s thesis witnesses.

In the first part of this paper, a structural characterization of ancient buildings is established by studying its material properties, its building systems and its failure mechanisms. Afterwards, the most common strengthening methodologies are reviewed, specifying its employability according to its purpose’s fulfilment.

For a successful design of seismic strengthening, the proceeding ought to be based on experimental and numerical studies. Thus, this paper mainly focuses on the assessment of ancient buildings’ non-linear behaviour. Therefore, in the second part of this paper, two walls of a 19th Century building in the city of Catania are idealized into equivalent frames. The numerical models designed are submitted to a non-linear Static Pushover Analysis and to a non-linear Incremental Dynamic Analysis.

The main purpose is to contribute to the development and discloser of a numerical model to easily and efficiently perform non-linear static and dynamic analyses of ancient masonry buildings, on current software (SAP2000®). The strengthening design ought to be based on this sort of analyses.

Keywords: ancient building; masonry; seismic retrofit; non-linear dynamic analysis.

1. Introduction

Non-linear analysis of ancient masonry structures in current software presents itself as a very interesting idea. Practical cases demand a quick approach which underlies computer models based on macro elements, applicable when the structure is composed by walls with such dimensions that the stress along the element may be considered as uniform. As the structure’s collapse is usually caused by wall’s in-plane failure mechanisms, two separate walls were modelled using an equivalent frame idealization in a proceeding often referred as SAM methodology. The selected walls belong to a building thoroughly examined by Italian universities for Catania’s Project purpose.

The models are submitted to a non-linear Static Pushover Analysis and to a non-linear Incremental Dynamic Analysis.

2. Structural Characterization

The knowledge of the structural materials properties ought to be a necessary condition for a successful seismic retrofit. Laboratory or in-situ experimental proceedings and visual inspections are of utmost importance.
Masonry is the main structural material in ancient buildings. It’s a heterogeneous, discontinuous and anisotropic material, with preferable rupture or stress surfaces. Mechanic properties are largely influenced by its forming materials and by constructive systems. Therefore each case has its own particularity. However, adhesion, cohesion and friction are always present in masonry. Another characteristic is the reduced tensile strength balanced by a better compressive strength. The shear resistance is translated by Coulomb’s law:

$$\tau = c_u + \sigma \tan \phi$$  \hspace{1cm} (1)

where, $c_u$ is the material cohesion; 
$\phi$ is the friction internal angle; 
$\sigma$ is the normal compressive tension.

On one hand, masonry behaviour under a monotonic push is generally described by multi-linear laws, often bi-linear [2]:

Under cyclic actions, masonry evidences hysteretic behaviour with loss of resistance after some load/unload cycles.

Wood elements are sometimes present in the walls of ancient buildings. Due to wood’s susceptibility to deterioration agents and to its frequent poor conservation, the structural properties are often compromised.

Masonry walls present two main types of failure mechanisms. The first one consists on out-of-plane damage, associated with rocking phenomena and resulting from poor linkage between orthogonal exterior walls. When the walls are well linked, the second type of mechanism occurs due to in-plane damage. Out-of-plane mechanisms are generally disregarded in buildings’ resistance assessment [12].

In-plane rupture of piers is usually due to sliding or diagonal cracking, even though it may be caused by rocking mechanism.

Resistance to sliding mechanisms is accounted for by analysing the next schematization: [4] [5] [13] [14]

$$V_{sd} = \frac{1.5 \cdot c_u + \sigma_0 \cdot \tan \phi \cdot D \cdot t}{1 + \frac{3 \cdot H_0 \cdot c_u}{\sigma_0 \cdot D}}$$  \hspace{1cm} (2)

where, $c_u$ is the cohesion; 
$\phi$ is the friction internal angle; 
$\sigma_0$ is the normal compression tension; 
$H_0$ is the distance from zero moment section to control section; 
$D$ is the wall width; 
$t$ is the wall thickness.
Diagonal cracking resistance is quantified by the equation proposed by Turnšek e Sheppard [15]:

\[ V_{id} = \frac{15 \cdot c_u \cdot D \cdot t}{\zeta} \left( 1 + \frac{\sigma_0}{15 \cdot c_u} \right) \] (3)

where the parameter \( \zeta \) establishes a correspondence between pier’s height and width, \( \zeta = \frac{H}{D} \) and \( 1.0 \leq \zeta \leq 1.5 \).

For quantifying rocking resistance it is necessary to disregard masonry’s tensile strength and the self weight of the pier, in order to have the same axial stress in both extremities of the pier. It is also necessary to consider the hypothesis of having a larger eccentricity in the bottom surface rather than in the top one:

\[ M_{id} = \frac{\sigma_0 \cdot D^2 \cdot t}{2} \left( 1 - \frac{\sigma_0}{k \cdot f_c} \right) \] (4)

\[ V_{id} = \frac{\sigma_0 \cdot D^2 \cdot t}{2 \cdot H_0} \left( 1 - \frac{\sigma_0}{k \cdot f_c} \right) \] (5)

where, \( f_c \) is the maximum compressive stress (designvalue);
\( k \) is a factor to assimilate the stress distribution to a rectangle (0,85).

The spandrel’s significance in seismic resistance is proportional to the number of stories in the building [12]. In spandrels, the rupture is usually due to shear and its resistance is often regarded as being owed to material cohesion.

\[ V_{id} = A \cdot c_u \] (6)

### 3. Seismic Retrofit

Analysis, diagnosis, design and execution are generally foreseen as the phases of the strengthening process. Analysis consists on the gathering of information on the original project and execution and on any maintenance proceedings which may have taken place through out the building’s life. The diagnosis consists on visuals inspections and experimental proceedings to rigorously evaluate existing pathologies. After this two stages, a decision on the pertinence of carrying on to retrofitting, to demolition or replacement of structural elements needs to be made. The design must consider efficient technologies suitable for the existing deterioration and compatible with the national normative background, without overlooking the associated costs of intervening and maintaining such solutions. In seismic retrofit, the supervision throughout execution is very necessary since discrepancies between the initial considerations for the design and the reality found on construction sites is quite common.

The designer awareness that he is intervening in a building with its own history ought to condition his activity. Therefore, the strengthening design needs to be carefully executed, making use of every tools and skills he possesses and with perfect knowledge of the frequent anomalies and solutions.
Naturally, traditional materials and techniques should be applied in order to maintain the essence of historical buildings. In this way the compatibility between the old and the new is assured. Since every now and then designers do make mistakes or the solutions are poorly executed, the intervention must be reversible in order to at least not worsen the initial condition of the building. Due to the historical value that most ancient buildings present and the associated cost of each intervention, the materials’ durability needs to be sufficient to avoid constant interferences in the building’s normal use.

There are many techniques available though they are not browsed here, since the main focus of this paper is the evaluation of structures’ resistant capacity.

4. Seismic Analysis

From the observation of the damage on walls, it’s known that these concentrate on the surroundings of the openings. Therefore, it’s considered to be valid the assumption that in the other zones of the wall, masonry behaviour is approximately elastic. From this starting point a deduction may be made to idealize a wall into an equivalent frame on which each pier is represented by a single macro element with a limited number of freedom degrees and united to other elements with rigid macro-elements.

In order to suppress the necessity of correcting some excessive simplifications on the model proposed in 1978 by Tomazevic, Magenes et al in 2000 responded with another formulation of an equivalent frame, usually referred by the acronym SAM since it was first developed for a numerical program with the same name. The main differences reside in the possibility of occurrence of distinct collapse mechanisms in a single element, such as rocking, diagonal cracking and shear sliding in the pears and shear failure in the spandrels. The use of these formulations in non linear analysis is not very common.

In Italy, Catania’s Project main purpose was to evaluate the consequences that an earthquake similar to the one occurred in 1693 would have today. The selected buildings were submitted to numerical and experimental proceedings by various teams of researchers from different Italian universities. The results present themselves as of great value for evaluating the quality of similar methodologies developed for other software. Laurent Pasticier, Claudio Amadio e Massimo Frangi com [14] seized this opportunity to test a SPO of an equivalent frame model in SAP2000. A similar procedure will constitute the starting point to this study.

Two walls from a 19th Century building were idealized into equivalent frames. Experimental proceedings took place in order to evaluate masonry’s mechanical properties. [14]

![Walls idealized into equivalent frames. [14]](image)
5. Static Pushover Analysis (SPO)

SPO consists in a monotonic increment of horizontal loading to evaluate a structure’s resistant capacity to seismic actions. A capacity curve is drawn from the data collected, relating the base shear stress with the horizontal displacement at a point on the last story. Each horizontal load increment allows attaining a point of the capacity curve.

Most regulation proposes the definition of an elastic response spectra and posterior adaptation to non linear behaviour. From the intersection of the spectrum with the capacity curve it’s possible to define a performance point or a target displacement. [1] [5] [10] [11] [13]

SAP2000 allows modelling material’s non linear behaviour through link/support elements or through plastic hinges. Since in SPO the load is applied monotonically and the advantage of using link elements resides in the possibility of modelling hysteretic behaviour under cyclic loadings, plastic hinges were used in SPO. Plastic hinges allow modelling yielding and post-yielding behaviour of frame elements by defining independent parameters for momentum, shear and axial loading.

An elastic-perfectly plastic behaviour was adopted for pier elements. For plastic hinges a rigid-perfectly plastic behaviour was assumed.

Other additional deformation levels may be defined, such as Immediate Occupancy, Life Safety and Collapse Prevention. None of these parameters influences the structure’s behaviour. [6]

The maximum momentum in piers occurs in its extremities so the plastic hinges are positioned there. On the other hand, when the pier mass is concentrated on the top section instead of distributing it across its length, the shear diagram is constant. This hypothesis results from the consideration that SAM software concentrates the pier’s mass in the top section. Approximately, shear plastic hinges are placed in the mid-height section.

As for momentum plastic hinges’ deformation, since the objective is to replicate the results attained by other researchers, a deformation equal to 0.8% of the effective height was adopted in concordance with Italian seismic regulation and Eurocode 8 [13]. For shear plastic hinges, the deformation was

Table 1: Masonry’s mechanical properties. [14]

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s Modulus E</td>
<td>1500 MPa</td>
</tr>
<tr>
<td>Distortion Modulus G</td>
<td>250 MPa</td>
</tr>
<tr>
<td>Specific Mass</td>
<td>1900 kg/m³</td>
</tr>
<tr>
<td>Compressive Strength fcd</td>
<td>2.4 MPa</td>
</tr>
<tr>
<td>Friction µ</td>
<td>0.5</td>
</tr>
<tr>
<td>Cohesion Cc</td>
<td>0.2 MPa</td>
</tr>
</tbody>
</table>

The effective height of the piers corresponds to the deformable extent of the element and it’s estimated from geometric correlations similar to the ones used in other equivalent frame methods. [8]

\[ H'_{\text{eff}} = h' + \frac{1}{3} D \frac{(H' - h')}{h'} \]  

where, \( h' \) – height after applying geometric correlations; \( H' \) – height from a story to another.
limited to 0.4% of the effective height for the same reasons presented before. [13].

No plastic hinges were placed in the spandrels since in the original study they were disregarded for some reason. If otherwise were to be considered, the spandrels would have elastic fragile shear behaviour with residual resistance. Plastic hinges would have rigid-plastic fragile behaviour where the elastic loading branch would be almost coincident with the plastic unloading branch, also presenting residual strength. The plastic hinges were to be placed in the spandrel’s mid-section.

The loads applied to the model are due to the walls self-weight and to the floor loading. The floor loading is estimated through the influence area methodology. Once again, as the objective is to replicate previously attained results, the Italian rules were applied in the estimative of the floor variable loading.

The horizontal equivalent loading was calculated for a triangular distribution:

<table>
<thead>
<tr>
<th>Level i</th>
<th>Wall A $F_i/F_{H,A}$</th>
<th>Wall B $F_i/F_{H,B}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 1</td>
<td>0.22</td>
<td>0.22</td>
</tr>
<tr>
<td>Level 2</td>
<td>0.40</td>
<td>0.37</td>
</tr>
<tr>
<td>Level 3</td>
<td>0.38</td>
<td>0.41</td>
</tr>
</tbody>
</table>

where, $F_i$ – horizontal loading in level $i$; $F_{H}$ – base shear.

To completely define the modelling, it’s necessary to estimate the yielding and collapse parameters of the plastic hinges. The initial stress $\sigma_0$ in the piers is obtained by simply applying structure’s vertical loading. Though, local collapse of elements changes $\sigma$ throughout the analysis. In the software SAM this parameter is automatically updated in the plastic hinge yielding stress definition. As SAP2000 does no support a similar function it is necessary to resort to a subterfuge in order to simulate such an effect. Therefore, two distinct analyses – PUSHOVER 1 and PUSHOVER 2 – were executed. In the first one $\sigma$ values are determined by simply applying vertical loading. In the second one $\sigma$ values will be calculated after the plasticization on the first hinge in the previous analysis.

For the limitation of plastic deformation it is necessary to determine elastic rotations. The following relation was assumed:

$$\delta_{plast} = \delta_{plast} - \delta_{elast}$$

$$\delta_{elast} = \frac{\delta_{elast}}{h_{eff}}$$

$$\varphi_{plast} = \varphi_{plast} - \varphi_{elast}$$

$$\varphi_{elast} = \frac{\varphi_{elast}}{h_{eff}}$$

$$\delta_{plast} = \delta_{plast} - \delta_{elast}$$

$$\delta_{elast} = 0.4\% \cdot h_{eff}$$

$$\delta_{plast} = 0.8\% \cdot h_{eff}$$

(8)

figure 9: Adopted displacement-rotation relation.[12]

The results were compared to the ones obtained by several Italian universities, particularly with the ones obtained by Pavia Universiteit and by Pasticier, Amadio and Fraiachino [14] who have followed a similar procedure.

The pushover curves and total horizontal load were the following:
6. Incremental Dynamic Analysis (IDA)

The seismic characterization considering the structure’s non linear behaviour must be achieved by using a time-history analysis. In an equivalent static analysis the gradual increment of horizontal loading allows to obtain a capacity curve. In an Incremental Dynamic Analysis, a similar result may be attained by performing a series of non linear time-history analysis for a seismic acceleration registry. The accelerogram is affected by different scale factors \( \lambda \). For each scale factor \( \lambda \) the maximum base shear and the maximum top displacement (which do not necessarily occur simultaneously) are measured and a point of the structure’s capacity curve is placed in an orthonormal referential. The capacity curve is drawn by uniting the points attained for each scale factor \( \lambda \). This proceeding is usually adopted for several different seismic registries (different earthquakes), with different characteristics and time durations.

The structure’s resistant capacity is evaluated using the seismic fragility concept, which is defined as the probability of reaching a specific limit state by solely analysing the intensity (PGA, for instance) of a known earthquake.

In SAP2000, the movement equation system is solved by modal superposition through Fast Nonlinear Analysis (FNA), which is an extremely quick and efficient method for structures with a limited number of non linear elements [6].

In FNA, non linearity is limited to link/support elements. Each link is composed by six springs, one for each degree of freedom, where different stress-deformation curves may be defined. Amongst the available links, it was

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**Table 3**: Maximum base shear by research group.

<table>
<thead>
<tr>
<th>Research Groups</th>
<th>Base Shear (H(\text{max})) (kN)</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wall A</td>
<td>Wall B</td>
</tr>
<tr>
<td>Génova</td>
<td>1673</td>
<td>617</td>
</tr>
<tr>
<td>Basilicata</td>
<td>1459</td>
<td>503</td>
</tr>
<tr>
<td>Laurent Pasticier – SPO 1 –</td>
<td>1340</td>
<td>476</td>
</tr>
<tr>
<td>Laurent Pasticier – SPO 2 –</td>
<td>1340</td>
<td>467</td>
</tr>
<tr>
<td>Pavia (SAM)</td>
<td>1100</td>
<td>467</td>
</tr>
<tr>
<td>Diogo Pereira – SPO 1 –</td>
<td>1082</td>
<td>432</td>
</tr>
<tr>
<td>Diogo Pereira – SPO 2 –</td>
<td>1075</td>
<td>430</td>
</tr>
</tbody>
</table>

The collapse mechanisms were the following:

- **Wall A**: Pre-yielding, Post-yielding, Collapse
- **Wall B**: Pre-yielding, Post-yielding, Collapse

**Figure 10**: Pushover curves.

**Figure 11**: Collapse mechanisms.
chosen the multi-linear plastic pivot model since it allows hysteretic behaviour. The loading degradation is defined by a set of parameters resultant from experimental observation that loading and unloading from any displacement level tends to specific points and that every stress-deformation curve tends to cross the elastic loading curve in a specific point. [9].

![figure 12: Multi-Linear Plastic Pivot Model. [6]](image)

Sap2000 uses a base loading curve and the parameters $\alpha$ and $\beta$ to define the link’s hysteretic behaviour. By simulating, in SAP2000, an experimental proceeding of a wall submitted to a cyclic horizontal loading, Pasticier, Amadio e Fragiacomo [14] obtained stress-deformation curves identical to the ones from the actual proceeding. This was achieved by changing the loading and unloading parameters until both curves were identical. In that wall and with that masonry, it was obtained $\alpha_1 = \alpha_2 = \beta_1 = \beta_2 = 0.45$.

![figure 13: Definition of the link parameters. [14]](image)

Experimental simulation is the best way to define the link parameters. Though, data referring to cyclic experimentation on Project Catania’s masonry walls is not available. Therefore, the parameters achieved by Pasticier, Amadio e Fragiacomo [14] were adopted.

The use of link elements is limited to shear collapse mechanisms, as the hysteretic behaviour is evidenced in shear experimental proceedings. The use of plastic hinges to simulate this mechanism is not available for FNA. The only viable option would be to use plastic hinges and solving the movement equation system by direct integration. Direct integration is a time consuming method that is not suitable for the sort of quick approach to dynamic non linear analysis intended. As shear failure mechanisms are the most common in ancient masonry buildings, rocking failure mechanisms are disregarded.

The links are placed as shear plastic hinges were in SPO, for exactly the same reasons. The yielding and post-yielding parameters are also the same. The only difference is registered in the post-yielding behaviour, where it is considered that a 30% decrease in shear resistance occurs for each 6mm of horizontal displacement.

![figure 13: Shear-displacement curve. [14]](image)

The real earthquake acceleration registries were collected from the European Strong-Motion Database, following the criteria previously explained. However, due to the structural characteristics of ancient masonry buildings, usually presenting a limited number of stories which results in an in-plane wall
vibration frequency typical of proximity earthquakes, most registries are of close range earthquakes.

Table 3: Selected Acceleration registries.

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Date</th>
<th>Station</th>
<th>PGA (m/s²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friuli 1</td>
<td>06/05/1976</td>
<td>Tolmezzo</td>
<td>3.098</td>
</tr>
<tr>
<td>Kocaeli</td>
<td>17/08/1999</td>
<td>Tosya</td>
<td>0.108</td>
</tr>
<tr>
<td>Campano Lucano</td>
<td>23/11/1980</td>
<td>Calitri</td>
<td>1.725</td>
</tr>
<tr>
<td>Friuli 2</td>
<td>15/09/1976</td>
<td>Buia</td>
<td>1.069</td>
</tr>
<tr>
<td>Tabas</td>
<td>16/09/1978</td>
<td>Boshroyeh</td>
<td>1.004</td>
</tr>
<tr>
<td>Manjil</td>
<td>20/06/1990</td>
<td>Tehran-Sarif University</td>
<td>0.324</td>
</tr>
<tr>
<td>Azores</td>
<td>23/11/1973</td>
<td>San Mateus</td>
<td>2.689</td>
</tr>
<tr>
<td>Gazli</td>
<td>17/05/1976</td>
<td>Gazli</td>
<td>7.068</td>
</tr>
</tbody>
</table>

The capacity curves and the fragility curves obtained are the following:

Figure 13: Capacity curves.

Figure 14: Fragility curves.

7. Conclusion

In Wall A, the capacity curve obtained in the SPO is similar to the one obtained by the University of Pavia using SAM software. Though, the difference between the maximum deformations is significant. Different criteria on the definition of the maximum admissible displacement may justify such a difference.

In Wall B, the base shear value obtained is 8% inferior to the one estimated by the University of Pavia. This discrepancy is due to different yielding values adopted for the failure mechanisms.

For each wall, a second analysis was preformed in order to simulate normal stress variation on the piers throughout an earthquake. However, after comparing the results with the ones of the first analysis this proceeding showed to be time and effort consuming and of reduced utility.
In both walls the failure mechanism is due to shear sliding in the upper floor. This is a common mechanism in ancient masonry buildings because of the reduced normal stress in the upper piers, reflecting in its shear strength.

SAP2000 allows a correct evaluation of the influence of non linear behaviour in the resistant capacity of idealized ancient masonry buildings' walls. The methodology introduced also presents an easy way to identify the location where damage concentrates. This aspect is of great value for seismic retrofitting as it allows evaluating where the strengthening will be effective.

The IDA's preformed confirmed the previously presented failure mechanisms. One conclusion to withdraw from these non linear dynamic analyses is the important role of the hysteretic behaviour on a building's resistant capacity. Another conclusion is the relation between the wall's h/b ratio and the failure mechanism's nature.

In Wall A, the low h/b ratio caused the collapse to present low displacement levels. Therefore, the maximum base shear obtained is closely linked to the non linear elements (links) initial strength. This is the reason why the majority of the IDA’s capacity curves tend to a specific value. The ones which present a different behaviour are affected by the earthquake's intrinsic characteristics.

In Wall B, the greater flexibility in comparison to Wall A implies greater displacements in the upper level of the structure. In Wall B, the failure mechanism receives an important contribution of the non linear elements' excessive deformation which results in distinct values of maximum base shear for each earthquake. This reflects the importance of considering hysteretic behaviour on a dynamic non linear analysis.

SAP2000 allows estimating the dynamic non linear response of an ancient masonry building. The interpretation of the results is not as straightforward as in SPO’s. The analysis itself is time-consuming, although previous practice contributes to speed up the process.

Concluding, the presented methodologies are of great value for ancient masonry buildings’ seismic strengthening.

8. References


