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PERFORMANCE-BASED ASSESSMENT OF THE ARSENAL DE MILLY OF THE MEDIAVEL CITY OF RHODES

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Abstract. *The paper focuses on the performance-based assessment (PBA) of the monument named «Arsenal De Milly» which is located in the Medieval City of Rhodes in Greece. It is an unreinforced masonry structure characterized by an important mass value and is founded on a soil profile that consists of soft and stiff soil layers. As known, usually PBA refers to the use of nonlinear static procedures. In particular, in the paper it has been performed by applying the procedure proposed in the framework of PERPETUATE project (www.perpetuate.eu) for the performance-based earthquake preservation of cultural heritage assets. The seismic response of the structure has been analysed by comparing the results obtained from different modelling strategies, in particular: (i) the finite element approach (through a 3D model using brick finite elements); (ii) the structural element modelling approach (through a 3D model based on the equivalent frame approach); (iii) the macro-block modelling one. Both linear and non linear analyses have been performed. Moreover, the results from some microtremor measurements addressed to the structural identification of the building have been used to calibrate the mechanical parameters to be adopted in the models in the elastic range.*

1 INTRODUCTION

This paper focuses on the performance-based assessment of “Arsenal de Milly”, an unreinforced masonry monument located in the Medieval City of Rhodes in Greece. In particular, the procedure proposed in PERPETUATE project for the performance-based earthquake preservation of cultural heritage assets has been applied [1] (funded by the FP7-Theme ENV.2009.3.2.1.1, www.perpetuate.eu).

PERPETUATE methodology adopts a displacement-based approach. A full methodological path for the assessment of cultural heritage assets has been proposed, which is based on three main steps ([1], [2]). The first one includes: 1) classification of the architectonic asset and contained artistic assets; 2) definition of performance limit states (specific for the cultural heritage assets); 3) evaluation of seismic hazard and soil-foundation interaction; 4) as built information (non-destructive testing, material parameters, structural identification). The second step is related to: 5) the definition of structural models for the seismic analysis of the masonry building and the contained artistic assets; 6) verification procedures. Finally, in the third step, rehabilitation decisions are taken and, if necessary, the second step is repeated for the design of strengthening interventions.

In the paper, the application of such procedure is presented focusing the attention on substeps 4), 5) and 6) and briefly summarizing the results on the other ones. In particular, seismic response of the structure has been analysed by comparing the results obtained from different modelling strategies, and in particular: (i) from the finite element approach (through a 3D model using brick finite elements); (ii) from the structural element modelling approach (through a 3D model based on the equivalent frame approach); (iii) from the macro-block modelling.

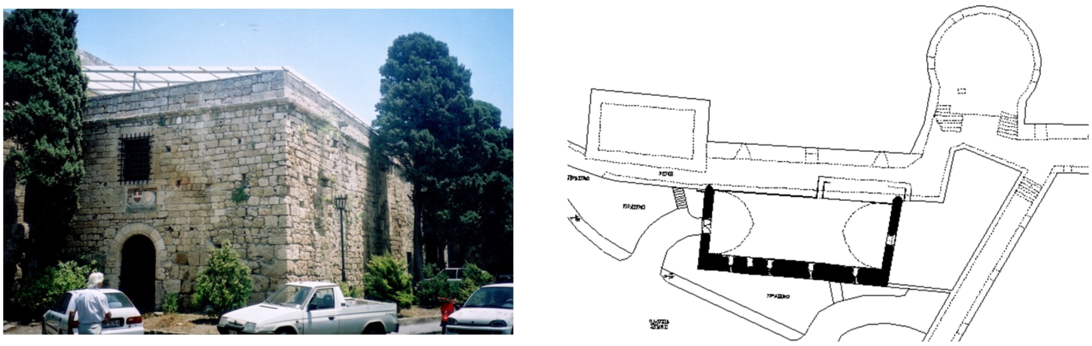
Moreover, results from some microtremor measurements addressed to the structural identification of the building have been used to calibrate the mechanical parameters to be adopted in the models in the elastic range.

2 ASSET CLASSIFICATION AND PERFOMANCE LEVELS CONSIDERED

Arsenal de Milly is located at the northeast corner of the medieval fortifications of Rhodes and it was built in the middle of the 15th century. It is a one storey rectangular building covered by pointed vaulted ceiling, and characterized by the presence of a very thick and massive fortified wall which supported one of its sides (Figure 1a). This monument was interested by many interventions in the past. Nowadays, it has been restored and the south wall is laterally supported by five buttresses (Figure 1b-c).

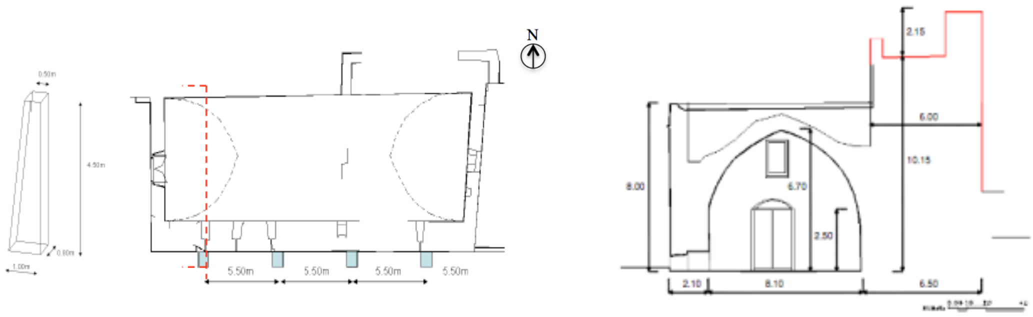
In the PERPETUATE project a proper asset classification of the cultural heritage has been proposed ([3],[4]). This classification is based on the prevailing seismic damage modes of assets and on the assumption that their occurrence is closely related to building morphology (architectural form, proportions) and technology (type of masonry, nature of horizontal diaphragms, and effectiveness of wall-to-wall and floor-to-walls connections). It is functional to the proper choice of the models to be adopted for the seismic assessment ([5], [6]). According to this classification, by taking into account the configuration of the building (which is quite simple and not characterized by a significant number of intermediate floors) and the information acquired through the in-depth as built information, it may be mainly classified according to Class B - *Assets subjected to prevailing out-of-plane damage*. This is consistent with the damage pattern, before the restoration too (Figure 2).

In PERPETUATE project specific targets and performance levels have been defined in order to consider the conservation and safety of people in an integrated approach ([3], [6]), and in particular they are related to: Use and human life (U), Building conservation (B); Artistic asset conservation (A). For each PLs the related earthquake hazard levels, expressed in terms of return period ($T_{RD,PLi}$, with i=1..4) have been indicated; these values maybe eventually modified through a proper coefficient in order to take into account the relevance of the asset ([3]). In the case of Arsenal de Milly (since the building is not used for public functions and not relevant artistic assets are present) the seismic assessment needs to be checked only with regard to the *Collapse Prevention* (Architectonic Asset Target, $T_{RD,PL3}$ =475).



(a)

(b)



(c)

Figure 1: The Arsenal de Milly before (a) and after (b) the restoration ([7], [8]); ground plan (on the left) and cross-section (on the right) after the restoration (c), with the buttresses marked in light blue.

3 AS BUILT INFORMATION

In this sub-step, geometrical, technological and mechanical features of the asset have been analysed in depth. Several data have been acquired related to: geometry of the building; foundations; mechanical parameters; historical data on transformation and damage; state of maintenance; dynamical behaviour ([9]).

From geometrical point of view, it is a building of rectangular plan and average dimensions 10.20m × 23.88m (Figure 1). It is covered by a vaulted ceiling. The vault has not the usual semi-cylindrical shape, but it is pointed at an average height of 6.70m. It consists of the original (bearing) arch at a height of 2.50m at its genesis and thickness 0.25m and has an overlying layer of mortar (kourasani) and an additional (non-bearing) thin "skin" of masonry (t=0.25m), while in the past it had been filled with a layer of clay material. The support for the vault is achieved monolithically at the south (outer) side with the 2.10m thick outer wall and at the northern tip (internal) is jointed and supported by a large volume of residue Wall of the Medieval Fortifications (thickness 6.00m).

The level of the foundation is at -1.90m with no widening. Arsenal De Milly has accepted many interventions in the past. Its damage pattern, before the restoration, proves the out-of-plane response. Cracks of width up to 8cm at the vertical walls and declinations at the top up to 10cm led to partial detachment of both vault and south wall (Figure 2). As aforementioned, nowadays, it has been restored and the south wall is laterally supported by five buttresses (Figure 1).

Figure 2: Pre-restoration damage pattern: detachment of the south wall (on the left) and longitudinal cracking on the vault (on the right).

In following paragraphs, some additional information related to the investigation plan aimed to the definition of masonry mechanical parameters and structural dynamic identification is illustrated.

3.1 Laboratory tests for the definition of mechanical properties of masonry

In situ and laboratory tests took place in order to determine (i) the constitution of mortars and stones (via chemists and mineralogical analyses) and (ii) the compressive strength of these materials (via core borings). The results show that the mortars are pozzolanic-lime mortars and the stones are sand-limes. The compressive strength of both materials of masonry is low.

More specifically, the pozzolanic-lime mortars have high content of fine aggregate (smooth surface, varying composition, natural origin) and its compressive strength varies from 3.14MPa to 4.56MPa. The stones used are local sandstones (lime stones) with compressive strength varying from 5.81MPa to 9.08MPa and tensile strength at around 0.75MPa.

3.2 Microtremor measurements for the structural dynamic identification

Figure 3 presents the location of the sensors for two different configurations: Figure 3a, the sensors 107, 109, 103 and 104 were placed on the top of the structure while the sensor 102 was placed in the ground surface close to Arsenal De Milly and Figure 3b, the sensors 107, 109, 108 and 104 are placed in the four corners of the structure while the sensor 105 was placed in an excavation inside the structure.

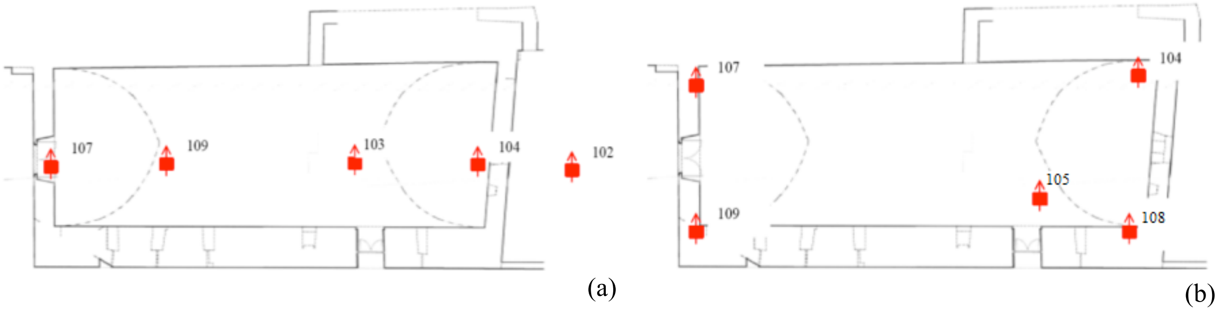


Figure 3: Sensors on the (a) longitudinal and (b) transversal direction

Figure 4a and Figure 4b show the results of the ambient vibration measurements that were performed considering the longitudinal direction ([10]). Figure 4a shows the Fourier spectral ratio given from the sensors situated on the corner near the defensive wall (107 and 104). On this Fourier spectral ratio the first peak appears at the frequency of 8Hz (0.125s) while the second amplification appears at 11.6Hz (0.086s). Figure 4b shows the Fourier spectral ratio given from the sensors placed at the centre of the vault (109 and 103). On this Fourier spectral ratio there is only one peak at the frequency of 8Hz (0.125s). These results verify that the structure is very rigid in the longitudinal direction where no torsional effects appear.

For the transversal direction, Figure 4c shows the Fourier spectral ratio for the sensors placed in the four corners of the structure (107, 109, 108 and 104). The sensor 109 placed in the south-east corner of the structure, present more clear peaks and the higher amplification compared to the other sensors (107, 108, 104). The first peak is at 5.1Hz (0.196 s) that is, in the same time, the predominant peak of the sensor 107 situated in the north-west corner of the structure. The second peak is around 8Hz (0.125s) and the forth peak is around 12Hz (0.083s). The last two peaks (8 Hz and 12 Hz) are also observed for the sensor 108 placed in the south-east corner of the structure. The third peak is at around 10Hz but it is not representative for the behaviour of the structure. The several peaks that appear in the Fourier spectra and confirmed in the Spectral ratio of the sensor 109, situated in the corner of the structure, suggest an important torsional effect. This effect, which is less amplified in the corner with the sensors 108, is due to the different rigidity between the structure and the adjacent defensive wall and due to the openings in the two transversal walls. The fact that there is almost no amplification for the sensors that are placed near the defensive wall (104 and 107) indicates that the structure does not work as a block and that the connections do not play an important role.

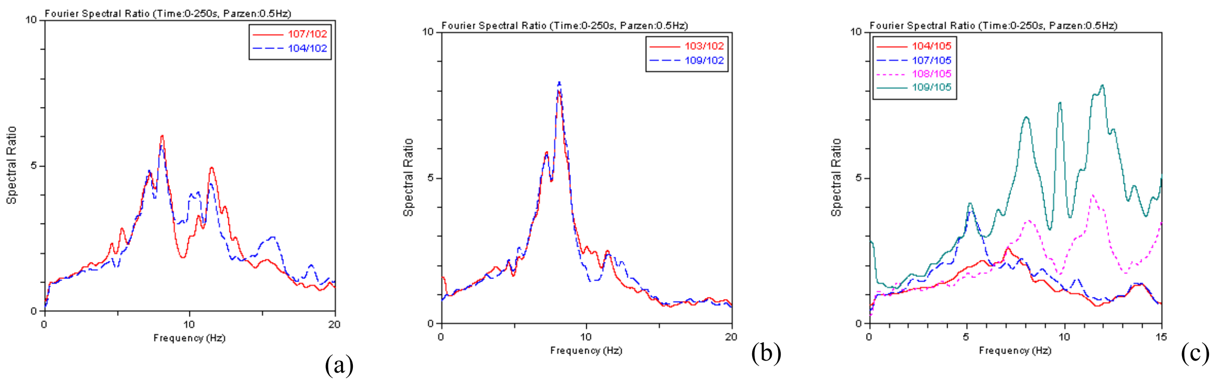


Figure 4: Fourier Spectral Ratio for the longitudinal direction – (a) sensors near the wall, (b) sensors near the centre of the structure and Fourier Spectral Ratio for the transversal direction (c)

3.3 Material properties

Based on the results obtained from laboratory tests and microtremor measurements, the structural identification of the building has been achieved.

Table 1 shows the mechanical properties and the masonry's strength parameters to be adopted in the models in the elastic range. These values have been also adopted in a technical report conducted in the Aristotle University in Thessaloniki [8]. Concerning the masonry's cracked pattern, an effective elastic modulus (E_{eff}) is assumed. To this end, the elastic modulus adopted herein is the one proposed in the abovementioned technical report (E=1800MPa) after a reduction equal to 50% (E_{eff}=900MPa).

Compression strength f_{wd}	Shear limit strength f_{vklim}	Tension Strength f_{wt}	Modulus Elastic E_{eff}	Poisson ratio v	Self weight γ
1.8 MPa	0.2 MPa	0.18 MPa	900 MPa	0.25	22 kN/m^3

Table 1: Masonry's properties

4 MODELING

According to the identified damage modes and asset types proposed in PERPETUATE ([3]), models are classified following two criteria ([5]): scale of discretization (whether material or structural element one) and constitutive modelling of masonry (whether continuous or discrete). Four types of models are identified: 1) Continuum Constitutive Laws Models, 2) Structural Element Models, 3) Interface Models, 4) Macro-Blocks Models. Depending on the different classes of heritage buildings (churches, palaces, towers, defensive walls) the choice of the mode proper model for the seismic assessment can change.

Usually, the MBM models can be mainly useful to perform the seismic assessment of those structures characterized by a prevalent out-of-plane response, as Arsenal de Milly. However, in the examined case, the seismic response is also strongly influenced by the interaction with the defensive wall, as the results of the structural dynamic identification have revealed. For this reason, the global response and the effects of this interaction have been also examined, by using different types of models, in particular CCLM and SEM models.

Following, the models used for the seismic assessment are illustrated in detail, referring to the above mentioned categories (CCLM, SEM and MBM).

4.1 CCLM approach

This model was developed with continuum constitutive laws for material properties. We assumed homogeneous material properties. The Continuum Constitutive Laws Model (CCLM) is simulated using the finite element method and its geometrical and material properties stem from the data obtained during the phase of knowledge. Old studies, field measurements and laboratory surveys were very helpful for the development of the final CCLM that was used herein.

Eight-node brick elements with 3 degrees of freedom at each node were used. The non-linear response is distributed to the whole structure by using Drucker-Prager plasticity law [11]. The Drucker-Prager yield criterion is defined in OpenSees code [12] by the following parameters: the bulk modulus k and the shear modulus G which are functions of the elastic modulus considered, the yield stress σ_Y and the frictional strength parameter ρ. In order to calculate the values of the first two parameters (k and G), the masonry's elastic modulus E is taken equal to 900MPa as previously mentioned. The Drucker-Prager strength parameters, frictional strength parameter ρ and yield stress σ_Y could be related to the Mohr-Coulomb friction angle φ and cohesive intercept c by evaluating the yield surfaces in a deviatoric plane as described by Chen and Saleeb, 1994 [13]. This relation is based on the shear strength criterion as it is expressed in Equation (1), where the shear strength with no compression is $f_{vk0}=c=0.2$MPa, the frictional coefficient is $\mu=\tan\varphi=0.4$ and σ_n is the compressive stress. The values of these parameters were also adopted in a technical report concerning the Arsenal De Milly [8]. The values used for the Drucker-Prager strength parameters are shown in Table 2. The mass density is equal to 22.4kN/m^3 and the model is considered fixed at its base.

$$f_{vk} = f_{vk0} + \mu \cdot \sigma_n \tag{1}$$

Bulk modulus k	Shear modulus G	Yield stress σ_y	Frictional strength parameter ρ
500 MPa	300 MPa	0.12 MPa	0.272

Table 2: Parameters for Drucker-Prager plasticity law

As a first step we developed an accurate model which is very close to the real structure, which in the following paragraphs will be named as '*global model*' (Figure 5). At this model we assume that the defensive wall is monolithically connected to the rest structure. The CCLM was simulated with OpenSees code.

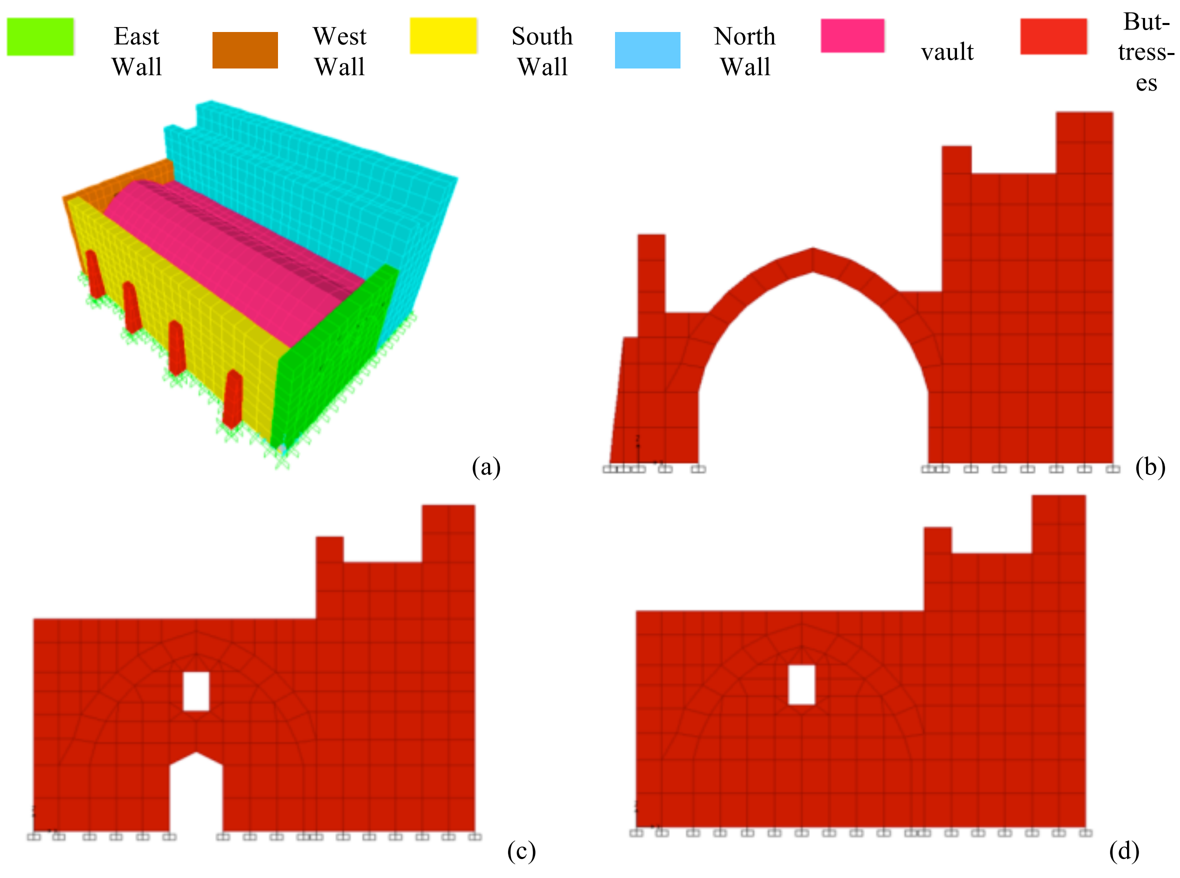


Figure 5: (a) 3D view of the spatial model with the 8 node brick elements (OpenSees), (b) 2D view of a typical cross section, (c) 2D view of the West Wall, (d) 2D view of the East Wall

Then, in order to compare results provided by the CCLM model with those of the Structural Element Model (SEM) developed in TREMURI [14,15] (illustrated in 4.2), in which the defensive wall has not explicitly modelled, the reactions of the defensive wall have been extracted from the base shear in both the directions (thus, by considering only the contribution of walls of Arsenal de Milly). Results related to this second case are called in the following as '*assumed model*'.

4.2 SEM and MBM approaches

In this case, the seismic analyses of the Arsenal de Milly has been performed by using in an integrated way two different types of model: a Structural Element Model (SEM), based on the Equivalent Frame approach (by using the Tremuri software, [14,15]) and a Macro-Block Models (MBM), adopted for the analysis of cross response of system vault-masonry walls (by using the MB-Perpetuate software, developed in the PERPETUATE Project [16]).

In particular, the analysis by the MBM model has been used in order to integrate the assessment of the cross response by including also their out-of-plane contribution, which is ignored in the global analysis (since one of the main hypothesis of the adopted SEM model is to consider only the in-plane response of walls) and which is considered very important in the examined case, due to its specific features (e.g. due to the presence of the buttresses and the high thickness of walls).

According to the SEM model adopted, each wall is discretized by a set of masonry panels (piers and spandrels), in which the non-linear response is concentrated, connected by a rigid area (nodes). Only the in-plane response of masonry walls is considered. Thus starting from the 2D modelling of walls, the complete 3D model is obtained by introducing also floor elements: in particular, they are modelled as orthotropic membrane finite elements [14]. For further detail see also [17,18]. Figure 6 illustrates: the structural model and the definition of the geometry of the structural elements (spandrels and piers) which form each wall in the Equivalent Frame Model; the Equivalent Frame Model of the two most relevant walls of the building.

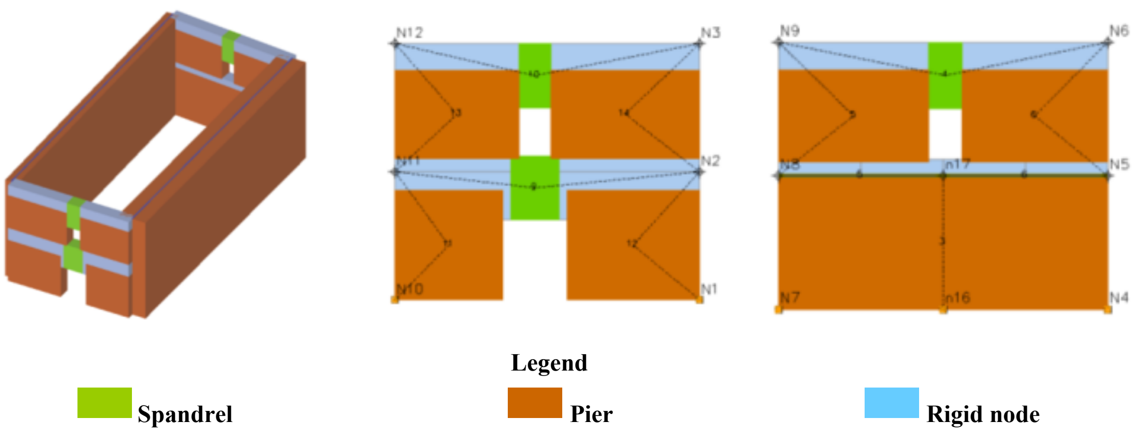


Figure 6: 3D view of the structural model (the visualization of the vault has been switch off) and Equivalent Frame Model of two walls of the building.

The analyses have been performed both in the X and in the Y directions. In the X direction, the effect due to the interaction with the defensive walls has been modelled by considering the hypotheses of the constrained horizontal displacement (Figure 7). In the Y direction, no constraints are considered: in this direction, in fact, it has been assumed that the Arsenal de Milly is probably added in a second historic phase, so that there is no adequate interconnection with the defensive walls; the analysis has been performed only in negative way by considering the constrain offered by the defensive wall in the other direction (Figure 7).

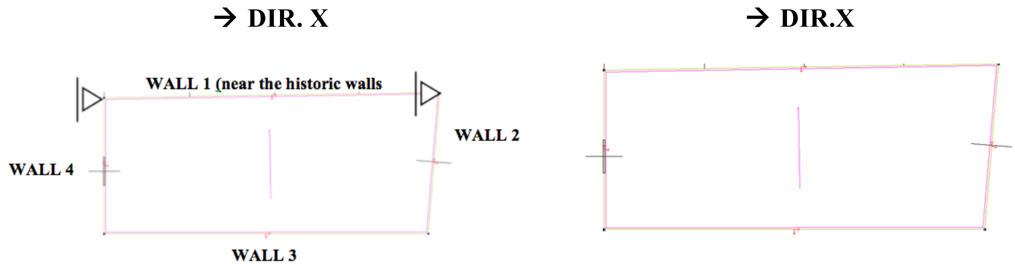


Figure 7: Constrains assumed in X direction.

The analyses have been developed by assuming a multi-linear constitutive laws for piers and spandrels elements by defining adequate drift and strength decay limits, differentiated as a function of the different failure modes that may occur (e.g. Rocking, Diagonal Cracking, Bed Joint Sliding or mixed modes as well) and progressing damage levels reached (like those shown in Figure 8). These laws have been recently implemented in the Tremuri software ([6], [19]). Moreover, the ultimate strength is computed according to some simplified criteria which are consistent with the most common ones proposed in the literature and codes (e.g. in Eurocode 8 [20] and in the Italian Code for Structural Design 2008 [21]) for the prediction of the masonry panels strength as a function of different failure modes which may occur.

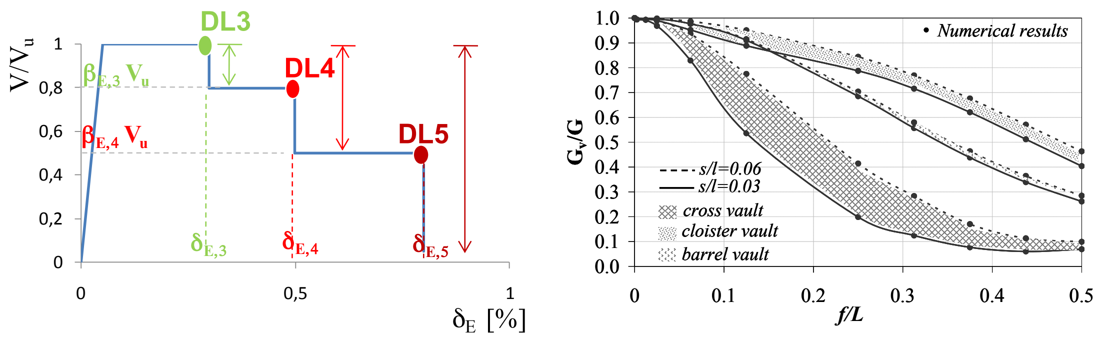


Figure 8: Multi-linear constitutive laws for piers elements (left, [6]); ratio Gv/G by varying the different types of structural vaults (right, [23]).

Table 3 illustrates the values of the mechanical parameters used in the analyses. The strength criterion used for piers and spandrels is a Mann & Muller one ([20]) expressed by eq. (2), with reference to the mortar failure domain:

$$\tau_u = \frac{c}{1+\mu\varphi_b} + \frac{\mu}{1+\mu\varphi_b}\sigma_y = \bar{c} + \bar{\mu}\sigma_y \tag{2}$$

where c and μ are the local cohesion and the friction respectively (assumed equal to 0.2 and 0.6) and ϕ_b is an interlocking parameter (assumed equal to 1).

Floor elements – in this case the vault - are modelled as orthotropic membrane finite elements, in particular: normal stiffness provides a link between piers of a wall, influencing the axial force on spandrels (E_{1v}); shear stiffness influences the horizontal force transferred among the walls (G_v), both in linear and non-linear phases. Regarding the modelling of the vault, it is important to underline that the definition of the equivalent stiffness properties assigned has been performed referring to the correlation laws proposed in [23] and illustrated in Figure 8, by considering the mechanical properties of masonry, adequately corrected through the use of specific coefficients obtained as a function of the geometrical features of the examined vault (which is similar to a barrel vault). In particular, a value of the rise-to-span ratio equal to 0.3 has been assumed by leading to a ratio of E_{2v}/E and G_v/G equal to 0.3 and 0.55, respectively; moreover, the ratio E_{1v}/E has been assumed equal to 1.6.

Mechanical Properties	
Masonry Panels	*Diaphrams*
$E_{crac.}$ = 900 MPa	E_{1v} = 270 MPa
$G_{crac.}$ = 300 MPa	E_{2v} = 1440 MPa
$\bar{c}$ = 0.125	G_v = 165
$\bar{\mu}$ = 0.375	t = 0.85 m

Table 3: Mechanical parameters associated to masonry panels (piers and spandrels) and to the vault.

The analysis of the cross response has been performed by using the MB-Perpetuate software [16], developed in the PERPETUATE Project. Figure 9 illustrates the analysed model (the initial configuration and the deformed one), where the plastic hinges are represented with an orange circle. In particular, the hinge n.1 at the base of the pillar on the left has been moved inside the section in order to take into account the buttresses' contribution. The position of the hinges has been defined on the basis of the expected response, evaluated from the observation of the recurrent seismic damage of vaulted structures and by taking into account the specific condition of constrains which characterized the examined case.

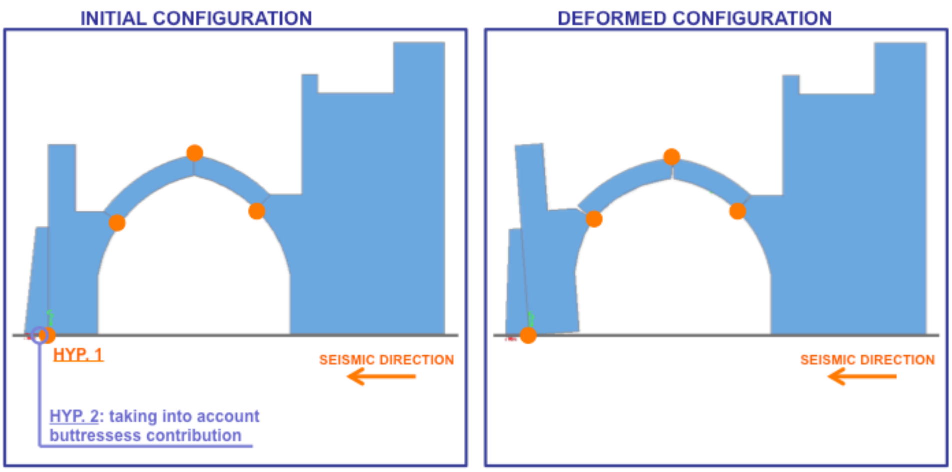


Figure 9: Identification of the collapse mechanism in case of MBM model.

5 SEISMIC ASSESSMENT

5.1 Seismic hazard

In this paragraph the appropriate seismic hazard for Arsenal de Milly will be presented. According to the results of past geophysical surveys and field measurements that conducted in the Medieval city of Rhodes, the studied monument could be classified to soil class C according to EC8 soil classification scheme. Moreover, the appropriate for the studied monument peak ground acceleration values stem from the results of Deliverable 24 [24] of the

PERPETUATE project, where a vector-valued probabilistic seismic hazard assessment (VPSHA) methodology was used for the estimation of a uniform hazard spectrum for Arsenal de Milly of Rhodes for three return periods (T_m=95years, T_m=475years and T_m=2475years).

More specifically, for Arsenal de Milly the seismic hazard is described in the following figures. Figure 10a and Figure 10b show the elastic acceleration response spectra and the elastic demand spectra respectively, for the PGA values proposed in D24 [24] and the three return periods. The assumed importance factor is equal to γ=1. The hazard curve, presented in Figure 11, shows the annual frequency of exceedance (λ_R) as a function of the peak ground acceleration PGA (a_g).

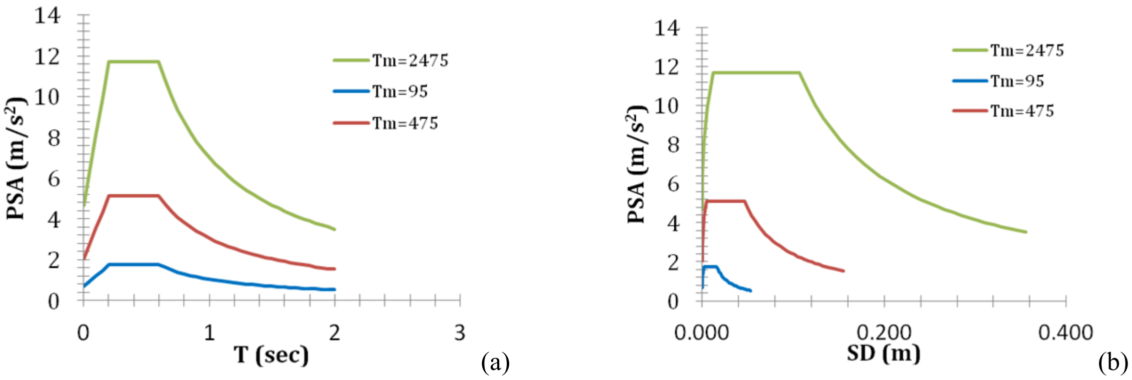


Figure 10: (a) Elastic acceleration response spectra for the PGA values according to D24, for three return periods (95, 475 and 2475 years) for soil class C according to EC8 for importance factor γ=1, (b) Elastic demand spectra for the PGA values according to D24, for three return periods (95, 475 and 2475 years) for soil class C according to EC8 for importance factor γ=1

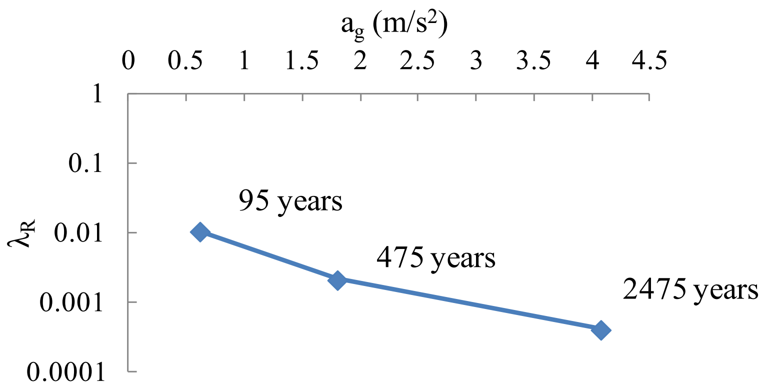


Figure 11: Hazard curve.

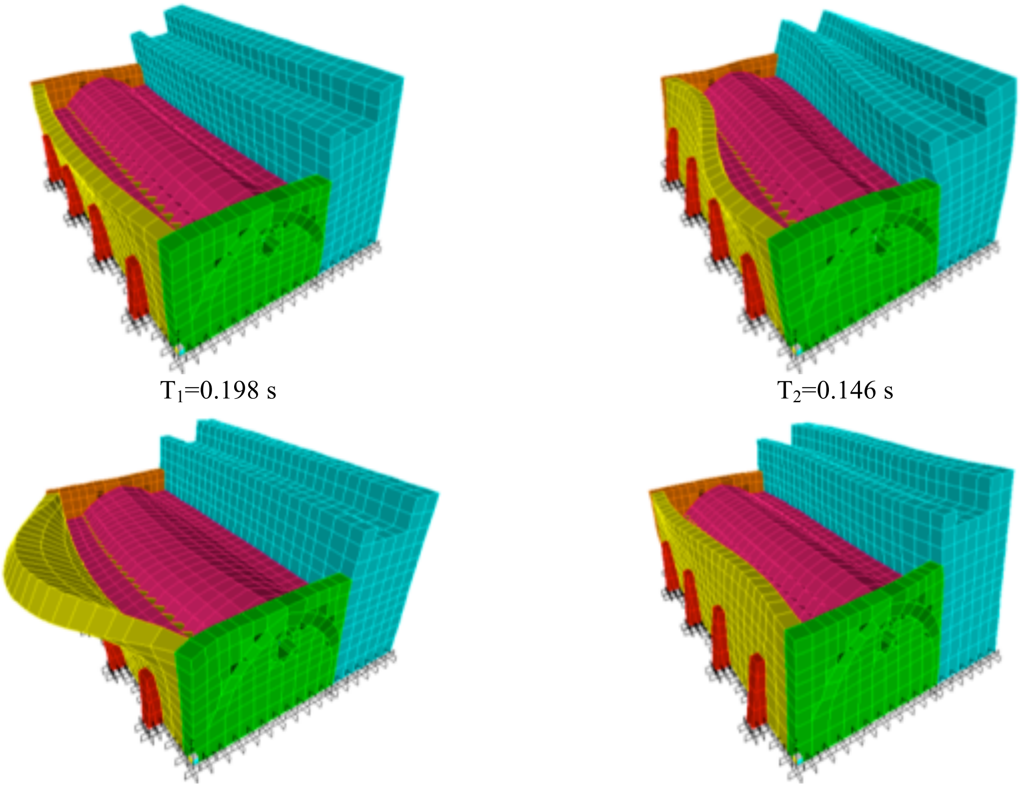
5.2 Results of modal analysis

In order to calibrate the elastic behaviour of the model in OpenSees, the modal analysis results in terms of period are compared with the values obtained by microtremor results. As it is indicated in Table 4, the microtremor results are verified for the fixed base model with a modulus elastic equal to 900 MPa.

The first and the forth modes of the model (in OpenSees) are the most significant as a high mass percentage is activated. In the first mode the predominant displacements are in the transversal direction (Table 5). The fundamental period is T_1=0.198s which coincides with the microtremor results Figure 4c). The second and the third modes activate a negligible mass percentage Table 5). In the forth mode the predominant displacements are in the longitudinal direction (Table 5). The fourth period is T_4=0.126 s which is in agreement with the microtremor results (Figure 4a and b). The modes shapes for the global model in OpenSees are illustrated in Figure 12 and their mass participation factor is indicated in Table 5. Moreover, concerning the rotational modes of vibration, it results that they are important and activate a high percentage of total mass at modes 1st, 2nd and 4th.

	Global model (OpenSees)	microtremor
$T_{transversal}$ (s)	0.198	0.196
$T_{longitudinal}$ (s)	0.126	0.125

Table 4: Comparison of the fundamental periods in the two directions between the model in OpenSees and the microtremor results.



T_1=0.198 s

T_2=0.146 s

T_3=0.142 s

T_4=0.126 s

Figure 12: Modes shapes and the period values obtained by OpenSees for Arsenal de Milly in the 3D model.

MODE	U_X	U_Y	U_Z	R_X	R_Y	R_Z
1	**60.70%**	0.00%	0.04%	0.02%	**23.65%**	**26.97%**
2	0.01%	0.03%	0.00%	0.00%	0.02%	**11.74%**
3	1.06%	0.00%	0.13%	0.08%	0.09%	0.34%
4	0.00%	**68.49%**	0.00%	**22.99%**	0.00%	**30.89%**

Table 5: Modal Participating Mass Ratios for the all the degrees of freedom, translational (Ux, Uy, Uz) and rotational (R_x, R_y, R_z).

5.3 Results of non linear static analyses and definition of performance levels

Figure 13 compares the results in terms of pushover curves as obtained by using the different modelling strategies already discussed (CCLM – *assumed model*, SEM and MBM). These curves have been obtained by performing non linear static analyses in both the seismic directions (in the case of CCLM and SEM models), while for the MBM model, a non linear kinematic analysis has been performed; in this latter case, the curve describes the progressive development of the collapse multiplier α (that induces loss of equilibrium of the system) for increasing finite values of the generalized displacement d, up to the value for which α(d)=0. Pushover curves of SEM and CCLM models are comparable. In general the base shear resulting from the CCLM model is higher than that obtained by the SEM one. Regarding the analyses in OpenSees, it is important noting that it has been possible to reach the convergence just until the part of the curve drawn with a continuous line, and then a suitable trend has been traced (illustrated with the dashed line in Figure 13). Furthermore, it has to be noticed that, referring to the transversal direction, the initial stiffness obtained with the CCLM model is higher; this effect could be associated to interaction between the structure and the defensive walls.

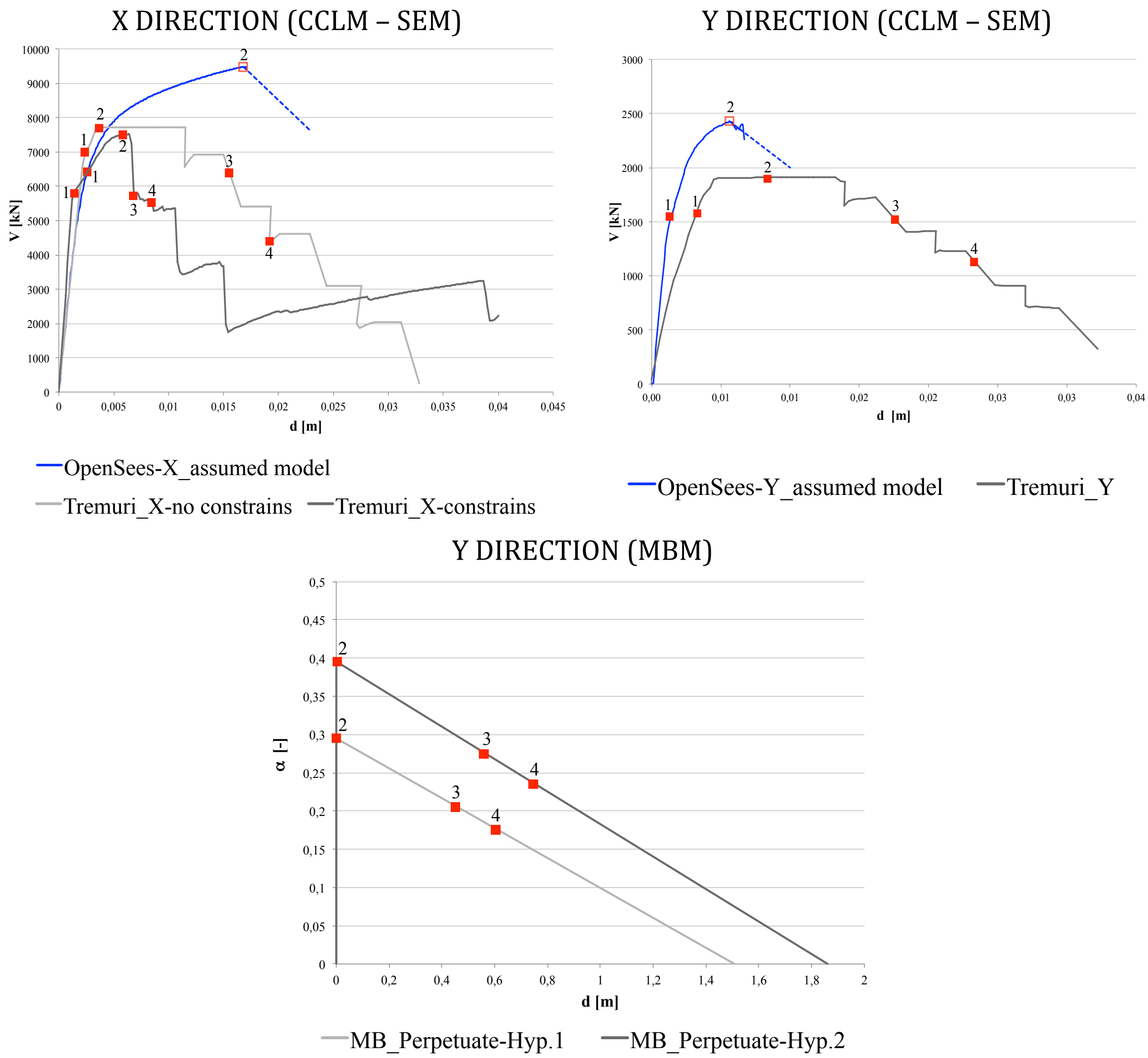


Figure 13: Comparison between the results obtained by using different models.

On each curve, the Damage Level (DL) – assumed coincident with the corresponding PL- are marked in red; they have been obtained by applying the multicriteria approach proposed in ([6], [25]). According to the multicriteria approach, the displacement on the overall pushover curve corresponding to the reaching of the *i-th* Damage Level (*DLi*, with i=1..4) is computed as:

$$d_{DLi} = \min\left(d_{DLi,E}; d_{DLi,M}; d_{DLi,G}\right) \quad i = 1,4 \tag{3}$$

where $d_{DLi,E}$, $d_{DLi,M}$ and $d_{DLi,G}$ are the displacements on the pushover curve corresponding to the attainment of the checks performed at element, macroelement and global scale, respectively.

Table 6 illustrates the limit thresholds assumed to define the damage levels at these different scales (e.g. in terms of drift δ_E, interstory drift δ_{is} or percentage of strength reduction β_E).

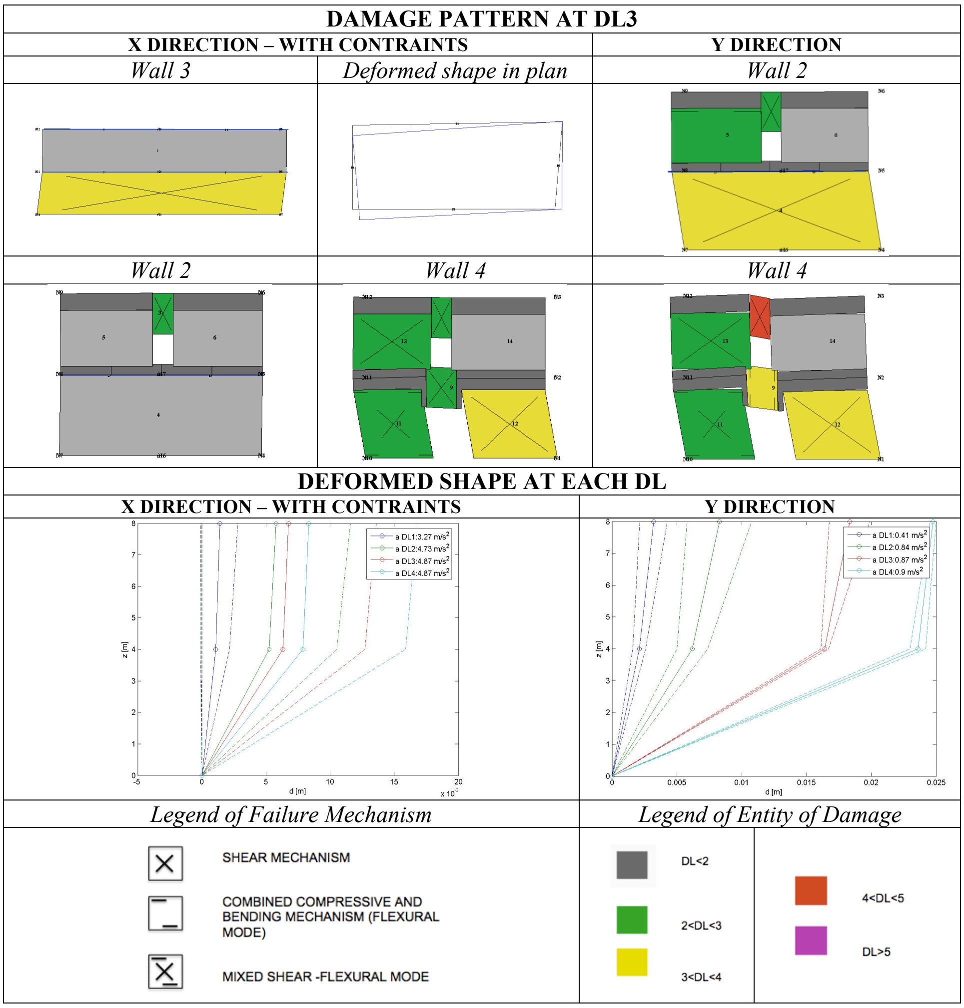


Figure 14: Damage pattern at DL3 of the most significant walls and deformed shape at each DL.

Figure 14 illustrates: the damage pattern at DL3 of the most significant walls (defined in Figure 7), as obtained by the analyses performed in Tremuri; the deformed shape at each DL. As it is possible to deduce by the damage pattern, in X direction the presence of the constrains determines a torsional effect, so that the orthogonal walls (walls 2 and 4) are mainly involved, while in the longitudinal walls only wall 3 is subjected to the seismic action: in fact, the horizontal loads toward this direction in the wall 1 are applied directly to the constrains. Obviously, in Y direction, unlike the walls 1 and 3, the walls 2 and 4 are interested by a quite relevant damage pattern, due to the fact that they are directed toward the seismic action.

DLi	Structural element scale Piers	Structural element scale Spandrels	Macroelement scale	Global scale
	-	-	$\delta_{is,1}$=0.1	-
	Reaching of maximum strength	-	Maximum base shear (V_M) $\delta_{is,2}$=0.3	-
	$\beta_{se,3}$ = 30 $\delta_{se,3}$ =0.3 (shear) – 0.6 (flexural)	$\beta_{se,3}$=50 $\delta_{se,3}$=0.3	$\beta_{M,3}$=30	$\beta_{G,3}$=20
	$\beta_{se,4}$= 60 $\delta_{se,4}$=0.5 (shear) – 1 (flexural)	$\beta_{se,4}$=50 $\delta_{se,4}$=0.6	$\beta_{M,4}$=60	$\beta_{G,4}$=40
	$\delta_{se,5}$= 0.7 (shear) – 1.5 (flexural)	$\delta_{se,5}$=2	-	-

Table 6: Definition of variables x at the different scales and limit thresholds (all limits are in %).

5.4 Safety verification

Figure 15 shows the comparison of results from all the models used (OpenSees – assumed model, OpenSees – global model, TREMURI- SEM model with constrains and MBM models) in terms of maximum acceleration ($a_{g,PLi}$) and return period ($T_{R,PLi}$) compatible with the fulfilment of PL_i (for both X and Y directions). The value of the seismic demand (in terms of PGA and return period) at PL3 is marked in Figure 16 with a red line. The values of $a_{g,PLi}$ have been computed according to the procedure proposed in PERPETUTATE [2], mainly based on the Capacity Spectrum Method [26] and the use of overdamped spectra.

The conversion of the pushover curve into Equivalent Degree of Freedom has been performed by using the Γ and m* factors (related to the assumed modal shape and the participation mass) as proposed by Fajfar in [27] and assumed also in Eurocode 8 [20]. The overdamped spectra have been computed by assuming the reduction factor proposed in Eurocode 8 [20], by computing the equivalent damping of the structure (ξ_{equ}) from the following expression [28]:

$$\xi_{equ}=\xi_{el}+\alpha\left(1-\frac{1}{\mu^{\beta}}\right) \tag{4}$$

where: ξ_{el} is the elastic damping assumed equal to 5%; α and β have been assumed equal to 20 and 1, respectively; and μ is the ductility value.

In the case of the results obtained in OpenSees, the values of a_{gmax} corresponding to PL2, PL3 and PL4 refer to the ultimate value obtained from the nonlinear analyses (marked by the white square in Figure 13).

As it is possible to deduce by Figure 15, the results obtained by the analyses performed in OpensSees (“assumed model”) and Tremuri (“with constrains”) are quite similar. Regarding the analyses in Y direction (where the prevalent structure seismic response is the out-of-plane one), the more reliable model is the MBM; these results are on the safe side with respect to those of “global model” by OpenSees; in this case, the seismic assessment obtained by using a SEM model is completely conventional (since the out-of-plane contribution is neglected). In

X direction, where the in-plane response is predominant, the SEM model seems adequately describe the seismic behaviour of the structure in comparison with the more refined CCLM model. In general, all the analyses determine results more precautionary than the values obtained in OpenSees with reference to the global model.

Finally, the seismic verification at PL3 is satisfied referring to the MBM model for the Y direction and referring to the other models for the X direction.

Figure 16 shows the safety factors referring to all the considered models in terms of maximum acceleration and return period, obtained by dividing the maximum compatible values of the structure by the design ones.

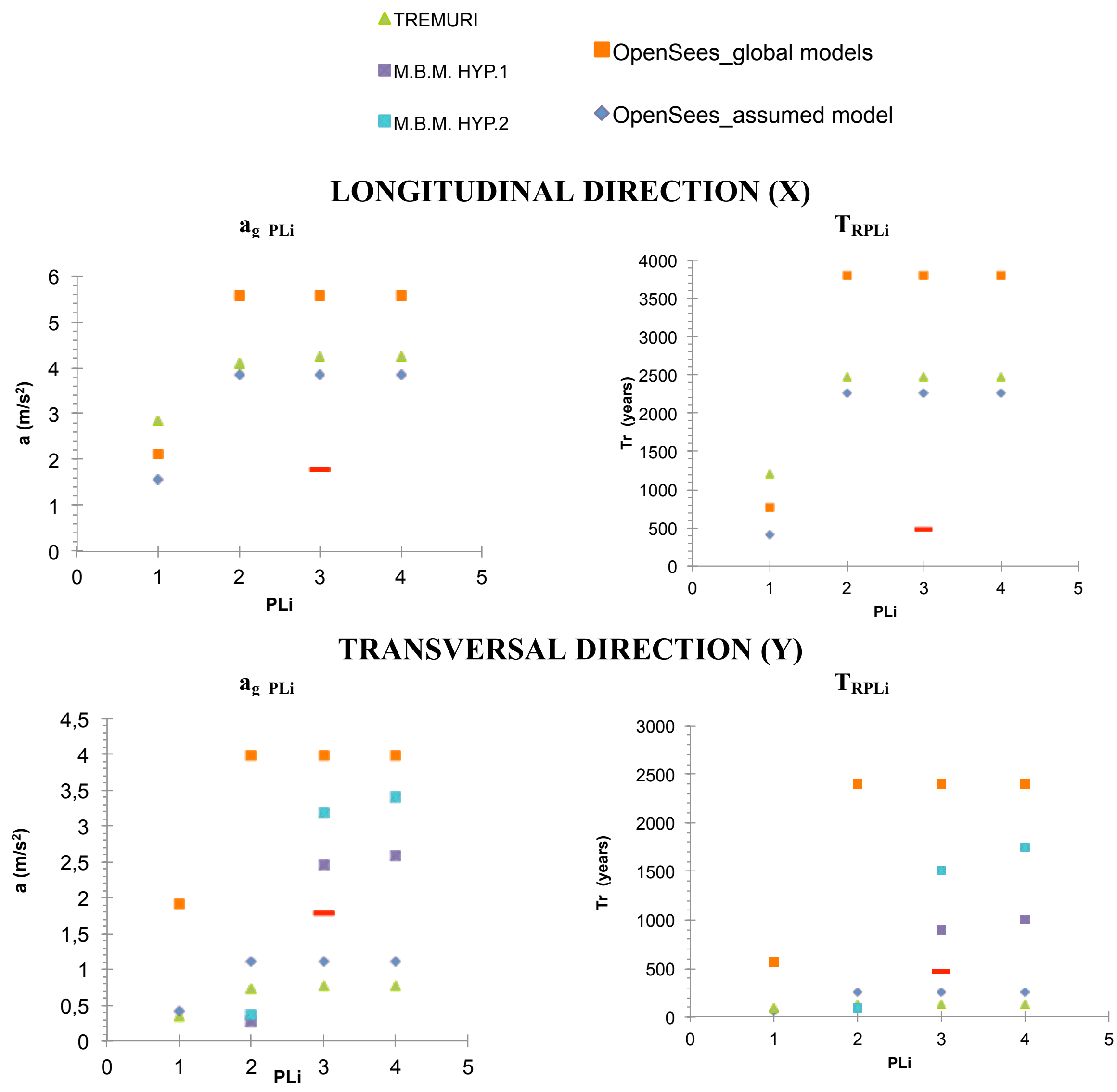


Figure 15: Comparison between maximum acceleration and return period at the four Performance Levels in the models used.

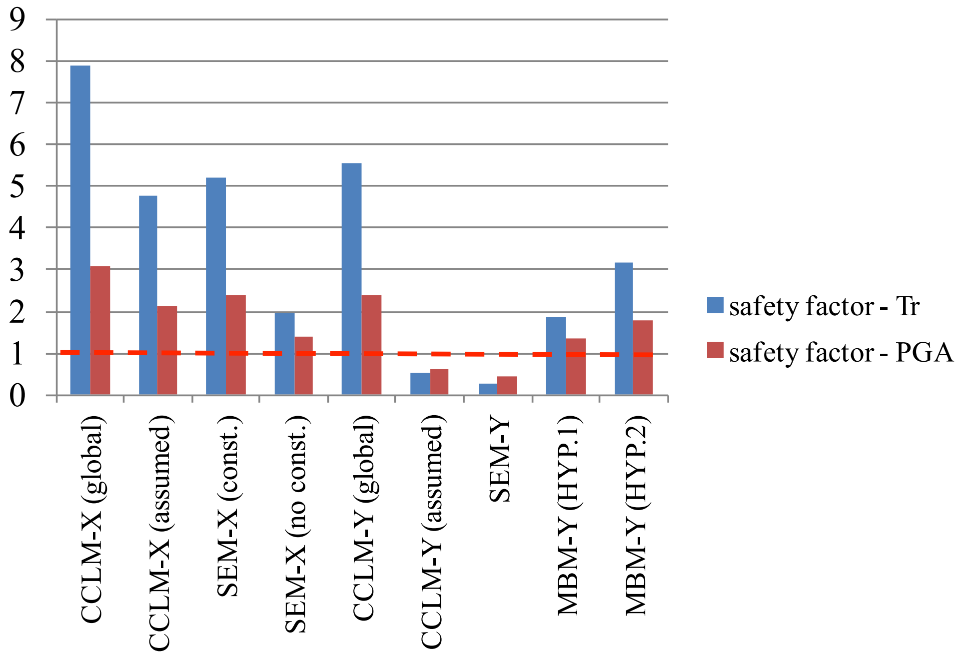


Figure 16: Safety Factors in terms of return period and PGA.

6 CONCLUSIONS

The paper presents the seismic assessment of the Arsenal de Milly according to the procedure proposed in PERPETUATE project, focusing the attention on results provided by different modelling strategies. Although the structure is quite simple from a geometrical point of view, its seismic response is quite interesting due to the interaction effects with the massive adjacent defensive wall. Calibration of models was supported by the results of ambient vibration tests that are very useful in such case in order to highlight torsional modes related to the aforementioned interaction effects. Seismic assessment results proved that the structure is able to fulfil a seismic demand associated to a return period of 475 years. In particular, the MBM (for the transversal direction) and CCLM and SEM models (for the longitudinal direction) seem the more adequate modelling strategies for a reliable assessment. Although the CCLM model is more refined than the SEM one, in non linear range it highlighted some numerical problems: thus, the combined use of the SEM model is useful to corroborate the results achieved. Finally, as future development, the soil structure foundation interaction has to be considered due to the specific feature of this structure (quite massive and founded on a soft soil).

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