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OBSERVATIONAL AND MECHANICAL MODELS FOR THE VULNERABILITY ASSESSMENT OF MONUMENTAL BUILDINGS

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SUMMARY

An overview of the available methodologies for the vulnerability analysis of the monumental buildings is proposed. If the analysis has to be developed on the monumental buildings at a territorial scale, the vulnerability models have to be referred to a typological classification and the approaches mainly belong to two different categories: observational and mechanical. In case of the most important monuments, more detailed models may be defined. Thus the methodology is organised in: a) Level 1, based only on typological studies on the observed vulnerability; b) Level 2, analysing a single part of the fabric (macroelement), considering simplified mechanical models suitable at the territorial scale; c) Level 3, defined on a more detailed analysis of the whole building or of a macroelement. In the level 1 model, the seismic input is represented in terms of intensity. On the other hand, the mechanical approach is based on the Capacity Spectrum Method that requires the definition of the Capacity Curve of the macroelement or of the whole building.

INTRODUCTION

In case of the monumental heritage, vulnerability and seismic risk are noteworthy topics. In seismic areas with high concentration of historic monumental buildings, it is worth to investigate these themes at a territorial scale, even if a more detailed study on some important monuments could be strongly significant. An assessment methodology can be developed through two different approaches: observational and mechanical, with different deepening levels.
After the recent Italian seismic events (Umbria and The Marches, 1997; Puglia and Molise, 2002), the damage survey of monumental buildings (in particular, churches) provided wide observational information (recurrent behaviour, damage patterns, intrinsic vulnerability, etc.). Through the data elaboration, important knowledge about churches in seismic areas was developed: reliable vulnerability models (Lagomarsino [1]) were elaborated, calibrated and tested.
The mechanical approach needs to be developed through simplified methods of analysis, such as collapse models, as proposed by Heyman [2]. In particular, the Equilibrium Limit Analysis provides a reasonably adequate description of the damage mechanisms in the masonry structures, even if under restrictions (the masonry considered as a rigid body with no resistance to tensile stresses; the earthquake, therefore, is simulated as a horizontal static force, proportional to the masses).

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On the other hand, in order to achieve a simulation of the seismic behaviour of the structure, a non-linear finite element simulation can be performed. In this case, some non-linear constitutive models for the masonry material are available (e.g. Gambarotta [3]). Simplified structural models, which take into account the masonry non-linearity, are also available (Galasco [4]).

DAMAGE ASSESSMENT TO CHURCHES AND MACROSEISMIC VULNERABILITY MODELS

From a systematic observation (Doglioni [5]) of the structural damage caused by the Friuli earthquake (Italy, 1976), the seismic response of churches may be described according to recurrent behaviour, traceable to the damage modes and collapse mechanisms of the different parts, called macroelements, which show an almost autonomous structural behaviour. Typical examples of macroelement are the façade, the bell tower, the apse and the side chapels. This approach, which is unsuitable for the analysis of complex buildings connected in a multiple way such as palaces or convents, allows a very effective qualitative interpretation for churches.

Damage survey and vulnerability of churches

In the ambit of the damage assessment to the churches struck by the Umbria and The Marches earthquake in 1997, the seismic vulnerability was studied and the main kinematic mechanisms of collapse in the different macroelements were singled out. Thus, a new methodology for damage and vulnerability assessment, Lagomarsino [6], was established; it is based on a survey form for churches, which considers eighteen indicators, each one representative of a possible collapse mechanism in a macroelement (figure 1).

2. DAMAGE AT THE TOP OF THE FACADE	
CRACKS IN THE TOP PART OF THE FACADE	
Facade weakened by wide openings Lack of a connection with the roof covering	

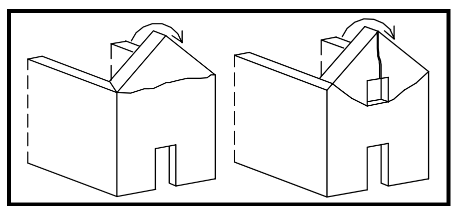

Figure 1. An example of damage assessment in terms of macroelements and collapse mechanisms.

The assessment of damage and vulnerability in Umbria and The Marches refers to more than 2000 churches, characterized by different typologies (from basilicas to small rural churches) and struck by different levels of macro-seismic intensity (from V to VIII of the MCS scale, according to the survey of INGV, the Italian National Institute of Geophysics and Volcanology) allowed us to test the vulnerability model connected to the survey methodology.

The statistical analysis of the data determined the Damage Probability Matrices (DPM) of the churches for four different macroseismic intensities. It is possible, therefore, to use these data directly for a priori forecast of the impact of an earthquake of given intensity on the cultural heritage. The damage histograms may be well represented by binomial distributions in terms of a free parameter, the mean damage grade μ_D. In figure 2, the increase of the mean damage grade with the intensity is shown for the whole set of churches (A+B) in Umbria and The Marches.

The survey form allows us to take into account the specific vulnerability of each church, evaluated through the vulnerability score i_v ($0 < i_v < 1$) described in Lagomarsino [1].

In figure 2, the mean damage values of the Umbria and The Marches churches are shown, dividing the samples of the churches into two sets: A (more vulnerable, $i_v > 0.4$) and B (less vulnerable, $i_v < 0.4$).

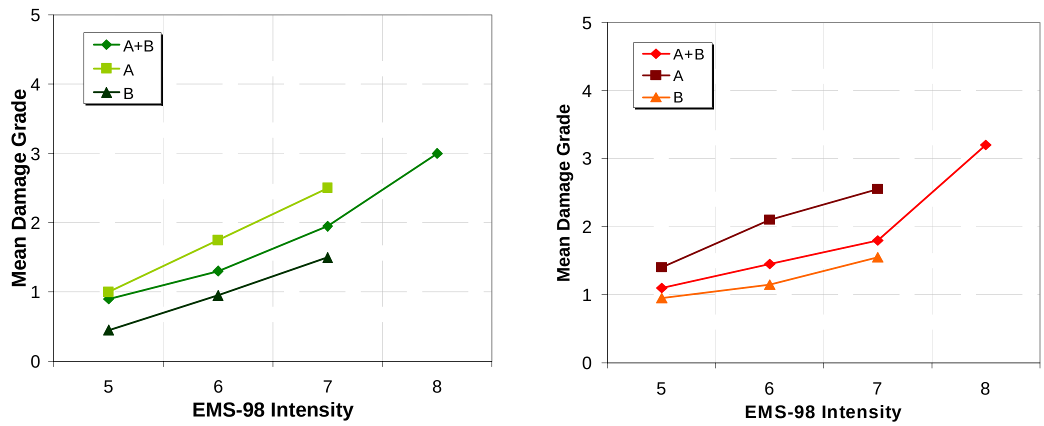

Figure 2. Mean damage grade of Umbria and The Marches churches.

If we associate the curve obtained for all the churches, to the vulnerability score value equal to 0.4 and the curves of the classes A and B respectively to the values 0.6 and 0.2 (mean value of the two classes), it is possible to define an analytic expression of the vulnerability curve:

$$\mu_D = 2.5\left[1 + \tanh\left(\frac{I + 3.4375 i_v - 8.9125}{\beta}\right)\right], \tag{1}$$

where μ_D is the mean damage grade depending on the parameter i_v, I is the macroseismic intensity and β =3.

Comparison between vulnerability curves of palaces and churches

Through a similar approach, a vulnerability index and vulnerability curves can be defined also in case of monumental palaces. This typology may be assimilated to the “massive stone” class in the ordinary building classification in Giovinazzi [7]. In fact, their construction is typically characterized by good quality materials and craftsmanship, especially because of their intrinsic importance and value.

The vulnerability model, even if derived from the EMS classification, is based on and validated through the damage observation in many ordinary buildings (in a homogeneous macroseismic area). In a few words, the method is based on the attribution of the vulnerability index (V_i) for a single building. It is defined on the basis of its typology and refined through behaviour modifier scores, related to some clearly detectable parameters (maintenance state, material quality, structural regularity, etc.). The model uses a very simple form and therefore represents a very quick tool and an instrument of immediate application to the territory.

The expression for the mean damage grade used in Giovinazzi [7], similar to the one proposed by Sandi [8], in case of the ordinary buildings is:

$$\mu_D = 2.5\left[1 + \tanh\left(\frac{I + 6.25 V_I - 13.1}{\beta}\right)\right], \tag{2}$$

where the parameter V_I is a vulnerability index, defined for ordinary buildings and varying from 0 to 1, according to the behaviour of vulnerability classes in EMS-98 and I is the macroseismic intensity.

This relationship, as previously remarked, may be proposed in case of the monumental palaces, using the parameter β (that controls the slope of the curve) equal to 2.3 (as in case of the “massive stone” ordinary buildings).

Moreover, we can notice that the relations (1) and (2) are analogous. Therefore, it is possible to use the same general expression, through the transformation of the vulnerability score i_v in the vulnerability index V_I provided by the following linear function:

$$V_I = 0.67 + 0.55 i_V . \qquad (3)$$

It is worth noting that the denominator in (1) and (2) is different; it controls the slope of the curves and it assumes different values to better fit the data obtained by the assessment campaign.
In figure 3, there is shown the comparison between the vulnerability curves of the churches (black lines) and those (grey lines) obtained for the "massive stone" regarding the classification of EMS-98 proposed. The comparison allows us to confirm the well-known higher vulnerability of the churches, defined by the higher values of the vulnerability index V_i. However, increasing the seismic intensity, the behaviour tends to become similar, due to the different values of the parameter β.

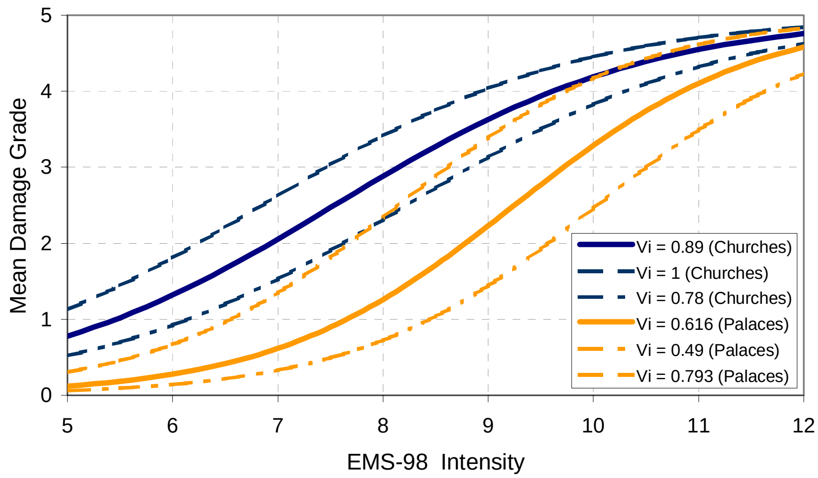

Figure 3. Comparison between massive stone buildings and churches vulnerability curves.

The vulnerability model determines a range of behaviour in the vulnerability curve, varying V_i (vulnerability index) that is defined in a certain value range and derives from the estimation made in the churches vulnerability form, which is based on a macroelement subdivision of the fabric.

THE VULNERABILITY ASSESSMENT METHODOLOGY

All vulnerability models, suitable for the territorial scale, have to be referred to a typological classification. These requirements arise also from the bibliographical studies in which a critical review of the methodologies was proposed. In particular, this analysis took into account different needs:
- the aim of making a survey and analysing a large sample of buildings, on a widespread territorial scale;
- the historical-architectonic value, that normally implies a more detailed approach, in comparison with those used in the vulnerability analyses on ordinary buildings;
- the requirement of the model validation, through the seismic damage actually surveyed after the earthquake.

Because of these aspects, the data collected through the survey have to be correlated with the constructive, typological and technological aspects, which have been highlighted by the seismic damage observation. In fact, the survey may show that some of these features turn out to be decisive in the vulnerability evaluation. Further parameters (such as age of construction, style, etc.) could be useful in refining the vulnerability model; in this case, they provide a starting point for a subdivision of the categories to be analysed.

The methodological review was carried out to organise the vulnerability analysis on three levels, through the definition of various vulnerability models, which represent a progressive deepening, based on the greater detail of the available data.
The proposed assessment methodology is organised in three levels:

- Level 1 is a macroseismic vulnerability model, in which the input is represented by the macroseismic intensity parameter and the vulnerability is defined by a vulnerability index V_i, related to qualitative parameters. It is based on typological studies, connected to the kind of the monument (palace, church, tower, etc) at a territorial scale;
- Level 2 is a mechanical vulnerability model, in which the spectral coordinates of the earthquake represent the input and the vulnerability is described by a capacity curve. The capacity curve, that describes the activation of partial collapse mechanisms in a single part of the fabric (macroelement), is defined by simplified models by means of a few geometrical and mechanical parameters, available at a territorial scale;
- Level 3 is a mechanical vulnerability model, in which the spectral coordinates of the earthquake represent the input and the vulnerability is described by a capacity curve, derived, if possible, by non-linear analyses. It is defined on a more detailed analysis of a single building, considered as a whole or through a macroelement, and so it cannot be applied at a territorial scale.

Level 1: inventory form, evaluation of the vulnerability index
Level 1 of the methodology is based on poor data; in this case the approach has to be necessarily typological, that means that the vulnerability is mainly determined by the typology of the monument. The classification of the monumental buildings is reported below:

- palace
- monastery
- castle
- church
- oratory/chapel
- mosque
- theatre
- tower/bell tower
- bridge
- urban walls
- triumphal arch
- obelisk
- statue/monumental fountain

The model individuated as the more effective and versatile is based on the attribution of a vulnerability index to each single building, defined in function of the typology of the monument and corrected through modifier scores, that are correlated to some easily noticeable parameters (state of maintenance, material quality, structural regularity, etc). The model is based on a very simple form and represents, therefore, a tool of quick and immediate application on the territory, which allows us to elaborate damage scenarios for each monumental typology.
The limit of level 1 methodology is due to the fact that vulnerability is considered in a global way; on the contrary, the damage observation has highlighted how, according to the architectonic complexity of such buildings, to the constructive characteristics (constructive phases, transformations, etc.) and to the poor tensile strength of the masonry, the damage and collapse often take place locally. An effective approach for studying the problem should be the construction decomposition into macroelements, parts of the building characterized by a substantially independent seismic response and simply associable to an architectonic element. However, a mean damage grade may always be defined.
The model is proposed for all the typologies previously listed. In some cases, the vulnerability derives from observed data (palaces and churches). In other situations, the value of the vulnerability index is based on expert judgements (it is related to the intrinsic vulnerability of the constructions).
The characterization of the behaviour of the different typologies was defined, partly, on a statistic basis (buildings and churches), and, partly, on an analogical basis. In particular, for the buildings and churches (Lagomarsino [9]), having at our disposition numerous statistic damage surveys for homogeneous subtypes, we defined the DPM. The matrices allow us to know, given a certain seismic intensity, the expected distribution of the damage level (figure 4), where the damage levels represent a quantitative

interpretation of the consequences caused by the earthquake on the structural and non-structural elements (cracks, deformations).

Mean damage grade μ_D:

0 – No damage
1 – Negligible or light damage
2 – Light damage
3 – Medium damage
4 – Severe grave
5 – Collapse

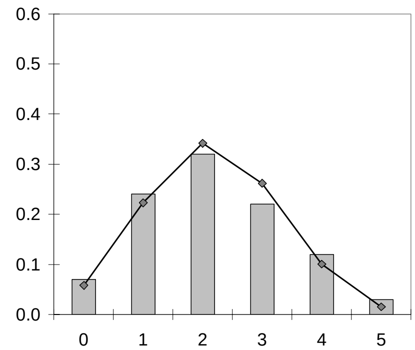

Figure 4. Mean damage grade μ_D and an example of damage grade distribution, function of a certain intensity (Umbria and The Marches earthquake, *churches* typology, intensity = VII).

As regards to the other typologies, as no specific statistical data are available, we chose to proceed by analogy concerning the two reference typologies (palaces and churches): a specific vulnerability index was attributed to each class and the slope of the relative vulnerability curve was defined. In Table 1, the vulnerability index values are shown.

Table 1. Value ranges of the vulnerability index for the different typologies.

TYPOLOGY	Vi-	Vi*	Vi+	β
Palaces/Buildings	0.496	0.616	0.956	2.3
Monasteries	0.616	0.736	1.076	2.3
Castles	0.356	0.456	0.766	2.3
Churches	0.77	0.89	1.26	3
Chapels/Oratories	0.65	0.77	1.14	3
Mosques	0.67	0.73	0.94	2.65
Theatres	0.616	0.736	1.086	2.65
Towers	0.636	0.776	1.136	2.3
Bridges	0.216	0.296	0.566	2.3
Walls	0.396	0.496	0.746	2.3
Triumphal Arches	0.376	0.456	0.706	2.3
Obelisks	0.396	0.456	0.746	1.95
Statues/Fountains	0.236	0.296	0.606	1.95

Secondly, the analysis took into consideration some specific aspects of the building. These are characteristic of the site (e.g. the position in the context), constructive features (e.g.: masonry quality) and maintenance. Moreover, we considered specific information about every single typology (e.g.: the presence of a raising façade in churches or the height of the tower).
The choice of the specific vulnerability parameters was made empirically, on the basis of the observation of the typical damage pattern. A modifying score, varying, in a limited range, the total vulnerability index of each single building, was defined and correlated to each parameter.

The information contained in the form
Besides general information (the accessibility and the possible crowding of the immediate surroundings), the form contains indication about general and specific vulnerability parameters.
The general parameters are vulnerability parameters common to all the typologies and they refer essentially to the state of maintenance of the building and to the transformations that it may have

undergone (Table 2). As these contribute to the determination of the total vulnerability index, fixed possible answers were predefined for every parameter. As already said, to each one corresponds a modifying score of the total vulnerability, defined on the basis of the typology. It is, therefore, necessary for the technician to fill in the form precisely.

Table 2. General vulnerability parameters and related modifying score values of V_I.

State of preservation	worst	+ 0.04
	medium	0
	good	- 0.04
Damage level	severe	+ 0.04
	light	+ 0.02
	none	0
Architectural transformations	yes	+ 0.02
	no	0
Recent interventions	yes	+ 0.02
	no	- 0.02

Masonry quality	yes	+ 0.05
	no	0
Site morphology	ridge	+ 0.04
	sloping	+ 0.02
	flat ground	0
Plan regularity	*It depends from the typology*	
Section regularity	*It depends from the typology*	
Position	*It depends from the typology*	

The specific parameters characterize the various building typologies. The same principle of the general parameters is valid. In Table 3 the church parameters are summarized.

Table 3. Specific vulnerability parameters and modifying score values: churches.

Plan regularity: nave typology	central	- 0.02
	one	0
	three	+ 0.02
Section regularity: raising elements or façade	yes	+ 0.04
	no	0
Position	included	- 0.02
	additions	+ 0.02
	isolated	0

Domes/Vaults	yes	+ 0.04
	no	0
Lateral walls height	low (< 6 m)	- 0.02
	medium (> 6 m and < 12 m)	0
	high (> 12 m)	+ 0.04

Level 2: determination of the capacity curve through simplified mechanical models

From the damage survey, it arises that, in case of a complex typology, the global behaviour could not be described in a proper way. In case of some monumental typology (tower, obelisk, etc.), the definition of a capacity curve describing the global behaviour of the monument is conceptually correct, even if of difficult realisation. As regards the churches there is another specific requirement, arising from the consideration that a capacity curve representing the global behaviour of the construction could be not totally correct. In this case, the curve has to be defined for a single macroelement. The damage mechanisms depend on vulnerability characteristics (e.g. the constructive and material aspects), specific of a particular building.

In order to develop the level 2 methodology at a territorial scale, there is the need of simplified analysis methods. The seismic vulnerability is evaluated through the Capacity Spectrum Method (Freeman [10]); the capacity curve of a single macroelement is determined through the simplified mechanical models. In level 2 models the capacity curve is defined in an analytical way by considering a few geometric parameters (e.g. in case of the triumphal-arch macroelement, we consider the arch thickness in crown, the span and rise of the arch, the pillar height, etc.)

The damage scenario is defined by the intersection (*performance point*) between the capacity curve and the response spectrum (*AD* format - *pseudo acceleration vs. displacement*) that is representative of the

chosen earthquake. The performance point has to be correlated to the damage levels previously defined. The analysis method used to define the capacity curve is based on the Equilibrium Limit Analysis. It takes into consideration kinematic theorem, applied to the masonry considered as an assemblage of rigid blocks, held together by compressive forces and liable to crack as soon as tensile stresses begin to be developed. This approach is based on the observation of the real behaviour of masonry structures. They are generally characterized by a negligible elastic deformation of the single parts, although displacements and rotations (due to the crack development) are possible. The earthquake, therefore, is simulated as a horizontal static force, proportional to the masses, and the obtained collapse multiplier λ represents the spectral acceleration. This approach allows us to estimate, with few geometrical and typological parameters, a macroelement capacity curve, estimating the effectiveness of some aseismic devices (tie-rods, buttresses, etc). The kinematic approach is operatively simpler than the static one. Once defined a set of possible mechanisms (a condition in which the structure may be represented by a kinematic chain of rigid bodies), the equilibrium is possible only under particular load condition. The value of the load multiplier λ for which the structure is in equilibrium is defined collapse (kinematic) load multiplier. The effective collapse mechanism is the one for which the load multiplier (that is the effective collapse multiplier) determines an admissible stress state (no tension) in the whole structure. For the limit-analysis kinematic theorem, the effective collapse multiplier is the minimum among all the kinematic load multipliers.
So, in order to be significant, the selected mechanisms have to be realistic and related to the observed damage patterns.
The first point on the capacity curve is defined trough the effective collapse multiplier λ calculated. In order to determine the other points on the capacity curve, we have to follow the development of the bearing capacity of the structure. A modified configuration of the elements (due to the mechanism evolution) is defined, the new multiplier is evaluated and associated to the correspondent displacement of the structure centroid. The last step is repeated until the value of the horizontal forces multiplier is equal to zero (displacements are such that the structure is not able to bear any horizontal load). In general, being S_u the theoretical ultimate horizontal displacement of the centroid, S_a the spectral acceleration and S_d the spectral displacement, the mean threshold between two different damage limit states may be so defined:

- **Limit state 1** (no damage) is defined by $S_a = 0.7\ \lambda$ (a fraction of the initial static multiplier).
- **Limit state 2** (slight damage) is defined by $S_a = \lambda$ (the initial static multiplier).
- **Limit state 3** (moderate damage) is defined by $S_d = 1/8\ S_u$.
- **Limit state 4** (extensive damage) is defined by $S_d = 1/4\ S_u$.
- **Limit state 5** (complete damage) is defined by $S_d = 1/2\ S_u$.

Level 3: determination of the capacity curve through detailed mechanical models
In order to study the seismic risk of monuments, the level 3 methodology presupposes that the vulnerability is studied through the Capacity Spectrum Method. The definition of a capacity curve representative of a single monument is needed, and, considering the detailing level in this methodology, the study can no more refer to a territorial scale. In a geographic area with homogeneous constructive features, it is possible to analyse in a detailed way a single monument, as a prototype of a typology.
In the case of other monumental typologies, a global structural response cannot be described, so the capacity curve has to be defined for each macroelement identified in the fabric (or for those that seem to be more vulnerable through an appraisal based on experience).
In order to define the capacity curve, we can use a refined method, such as the Finite Element Method or other non-linear structural models, specifically developed for some monumental typologies (Lagomarsino [11], Resemini [12]). In a Finite Element Analysis, the structure can be modelled through elements with a non-linear behaviour (Gambarotta [3], Podestà [13], Calderini [14]), which takes into account the limited tensile strength of the material and the progressive degradation of the stiffness beyond the pre-chosen

value of the failure stress. Different ways of failure (sliding, crushing, shear failure, etc.) may be considered.
In order to achieve a simulation of the seismic behaviour of the structure, a non-linear static incremental finite element simulation, considering large deflection, can be performed. An increasing horizontal acceleration represents, from a static point of view, the seismic force actions.
In the Finite Element Analysis, the continuum model allows us to consider structural deformability and non-linearity. It is a proper way to evaluate a realistic pattern for the cracks, accounting for the presence of elements like vaults, etc. Moreover it takes in consideration the influence of the successive load steps (due to the presence of plasticity phenomena, connected with friction effects). A disadvantage of this kind of analysis is the trouble in determining a convergent solution, leading to a high number of equilibrium iterations. Other important problems occur in getting the softening phase, consequent to the compressive and shear failure, in which the material progressively loses strength. In any case, taking in account large displacement, the softening phase is present, even if we only consider the tensile failure. In this condition, in order to achieve this behaviour, the geometric non-linearity option may be activated: under load control, the methods used are the arc-length, etc., but some troubles are still present.
Using Finite Element Models, it is possible to define the damage limit states not arbitrarily on the capacity curve (e.g. as a fraction of the ultimate displacement), but associating them to peculiar transition phases in the structural behaviour (e.g. the reaching of the tensile or compressive strength in some masonry parts or of a predefined limit value of the internal damage parameters).

EXAMPLES AND APPLICATIONS

In the ambit of the RISK-UE Project "An advanced approach to earthquake risk scenarios with applications to different European towns" (Contract: EVK4-CT-2000-00014, funded by the European Community within the 5th Framework Programme), the vulnerability of the monuments in the urban area of Catania (Southern Italy) was studied. In order to apply the level 1 methodology, the monumental heritage was survey at a territorial scale. The level 2 analysis was subsequently developed, considering one particular macroelement (the triumphal arch), which seemed to be particularly vulnerable, in the churches in the city of Catania.
The level 3 methodology was applied to the transversal response of the Church of S. Maria del Mar (Barcelona, Spain); also in this case, the study was developed in the ambit of the RISK-UE Project.

Level 1: vulnerability survey and damage scenario for the monuments of Catania

The total amount of the monumental buildings in the city of Catania (Southern Italy) is 198; the typological subdivision, in figure 5, highlights that churches and palaces represent the most recurrent classes. In about a week, the quick survey catalogued 150 monumental buildings, using the level 1 methodology form, described in § *Level 1: inventory form, evaluation of the vulnerability index*. From the data collected, we evaluated the vulnerability index of each monument.

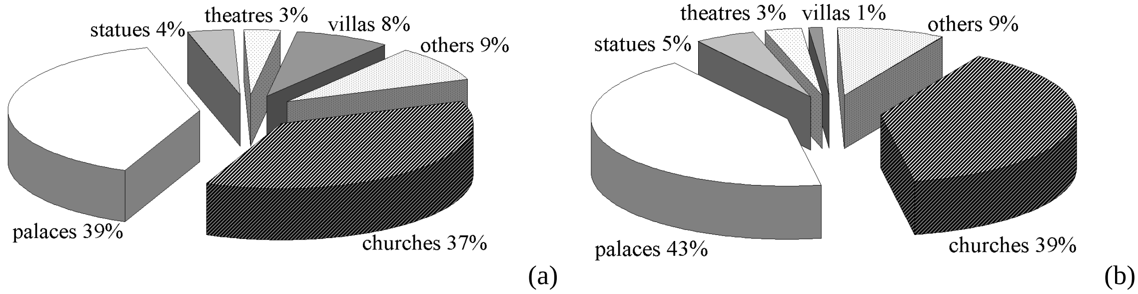

Figure 5. (a) Typological subdivision of the monumental heritage in the city of Catania; (b) Typological subdivision of the monumental heritage surveyed.

Analysing the most frequent typologies at a deeper detailing level, through the score-modifier survey, it was pointed out that the palaces have a medium-good state of maintenance, limited crack patterns, and good masonry quality. This can be explained because of their cultural and social importance. It is worth noticing that over 70% of the palaces present plan and section regularity and that only 30% were subjected to architectonic transformations or recent interventions.
As regards to the churches, the global result of the analysis is similar: a better masonry quality (related to the care in the material choice and craftsmanship) is noticed, but, in a few per cent, the state of maintenance is worst and the damage level is extensive. These situations are generally present in small churches, in the suburban area. It is worth noticing that about 80% of the churches have a single nave (notwithstanding the wide plan dimensions of many constructions).
Analysing the surveyed data and the vulnerability index, two damage scenarios, related to the reoccurrence of the historical earthquakes in 1693 and 1818, were defined.

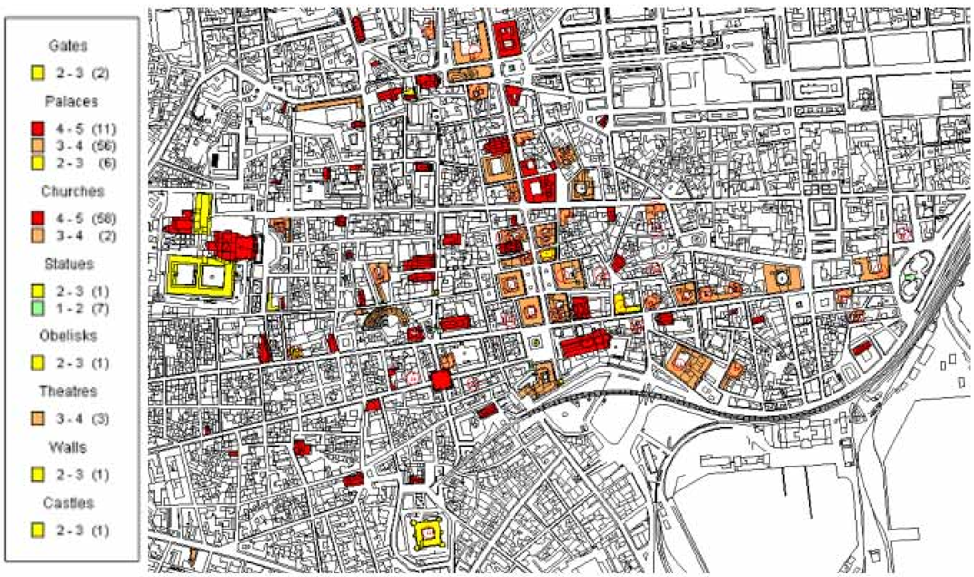

Figure 6. Damage scenarios in Catania in terms of μ_D (mean damage grade): 1693 earthquake.

In 1818 damage scenario, it could be noticed that the churches represent the most vulnerable typology; in fact they are the only buildings for which a significant mean damage grade μ_D is predicted (2-3 and 3-4). Similar observations can be formulated observing the 1693 damage scenario (figure 6). The seismic event was more severe than the 1818 one, and this condition causes a high damage level in a lot of buildings in all classes. In the other typologies the damage level is severe, whereas in 95% of the churches the predicted mean damage grade μ_D corresponds the structural collapse (4-5).
In Table 4, the number of damaged monuments in Catania in case of the 1693 and 1818 earthquakes is summarised.

Table 4. No. of monuments suffering seismic damage in the city of Catania.

No. of monuments		Damage level				
		1	2	3	4	5
Earthquake	1693	0	7	12	61	69
	1818	66	27	53	3	0

Level 2: capacity curves and damage scenario for the triumphal arches of the churches in Catania
In order to develop a seismic assessment methodology for the triumphal arches in Catania, the churches in which this macroelement is present were analysed. The damage mechanism is possible if the church is not

completely surrounded by other buildings. The churches with these characteristics are 21 (17 with a single nave and 4 with three naves). The evaluation of the damage scenarios was based on this stock.
Having observed the real damage patterns, it was possible to define the most probable mechanisms, and so the kinematic approach of the Equilibrium Limit Analysis was developed. For the triumphal arches of the single-nave churches were recognised two mechanisms (figure 7-a), involving the rotation of both the pillars (mechanism A) or only one pillar (mechanism B). For the triumphal arches of the three-nave churches the most probable mechanism (figure 7-b) was defined considering the kinematism of 7 rigid blocks (10 hinges).

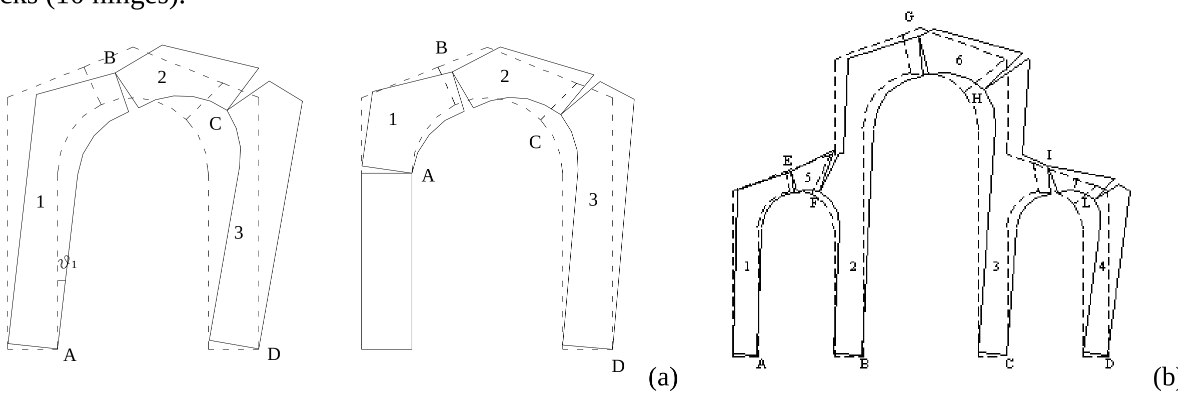

Figure 7. Possible kinematisms in triumphal arches: (a) single-nave church; (b) three-nave church.

In order to quantify the effectiveness of the metallic transversal tie-rod in the arch, the collapse multiplier and the capacity curve were evaluated both with and without this aseismic device. Several values of internal force in the tie-rod were considered. The results highlighted that the introduction of a tie-rod increased the load multiplier that activates the kinematism (due to the initial value of the internal force), even if in different percentage in relation to the shape of the arches (figure 8).

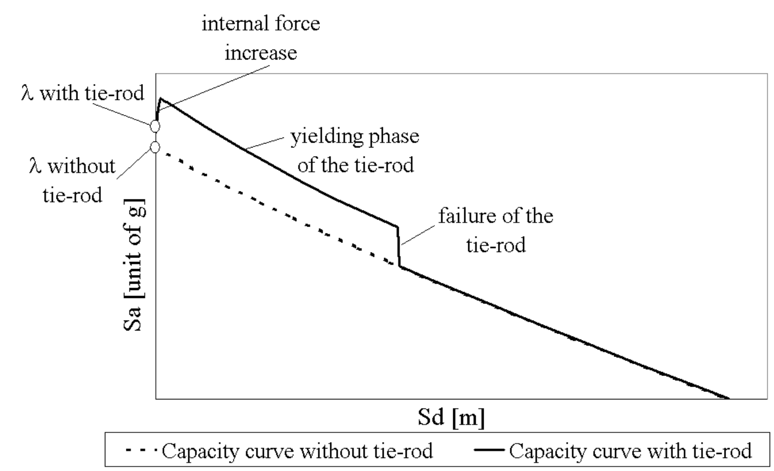

Figure 8. Capacity curve behaviour with or without the tie-rod.

As the horizontal displacement and the rotation increase, the capacity curve shows an increasing behaviour due to the force increase in the tie-rod, because of its elongation and function of its stiffness.
When the horizontal displacement makes the tie-rod reach the yielding state, the contribution cannot increase anymore, and it will remain constant until the displacement value that leads to failure (assumed as the 5% of the initial length of the tie-rod). Beyond this level there will be no more contribution, and the curve shows a sudden descending branch. The effectiveness of the tie-rod is mainly related to an increase of the initial collapse multiplier.
In order to determine the response spectra for a correct evaluation of the performance point, we had to

consider that the value of the structural damping increases as the damage goes on. We consider over-damped spectra, because, in this study, the structural ductility definition is not meaningful. Nevertheless, in this condition, the rocking mechanism is predominant, and so the structural damping do not increase very much; ranging values of ξ from 5% to 15% are used.
Using previously defined damage limit states (generally as a fraction of the ultimate displacement of the centroid), the damage scenarios for the triumphal arches were determined (with and without the tie-rod). The results are shown in figure 9.

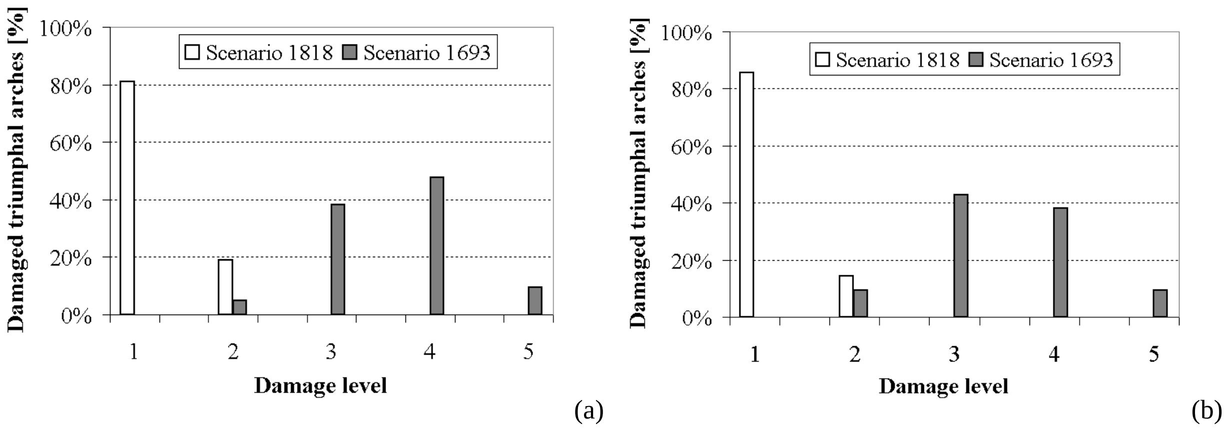

Figure 9. Damage scenarios for triumphal arches: (a) without the tie-rod; (b) with the tie-rod.

We can notice that the tie-rod represents an efficient retrofitting intervention, both in order to contrast the kinematism activation and, if the mechanism is activated by the earthquake, to limit the damage level.

Level 3: *FEM* simulation of the transversal response of the Church of S. Maria del Mar (Barcelona)
In this paragraph, an example of application of the level 3 methodology is proposed (Irizarry [15]). The monument studied is the Church of S. Maria del Mar (figure 10), in Barcelona (Spain).

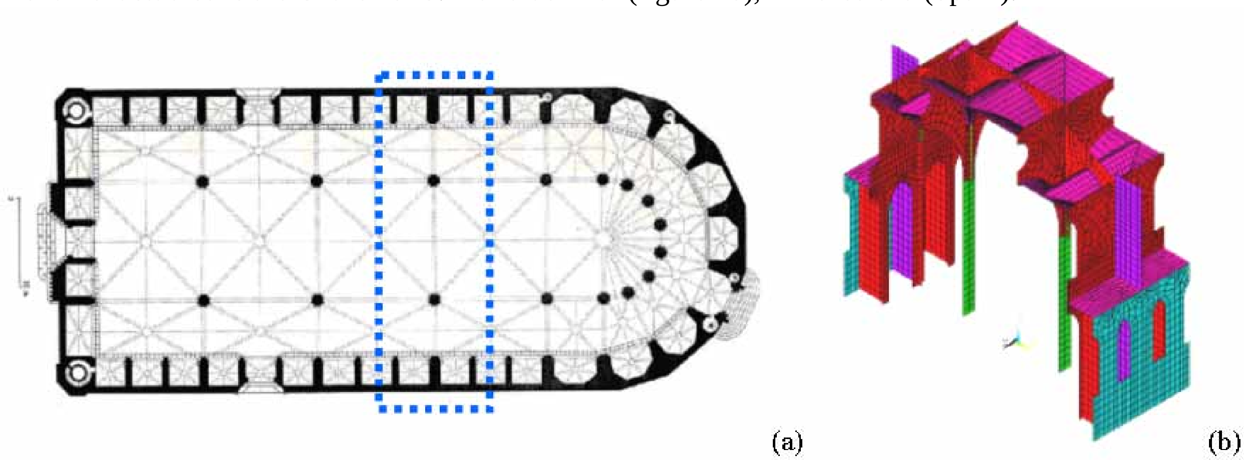

Figure 10. (a) Plan view of the Church of S. Maria del Mar and the macroelement for the transversal response; (b) *FEM* model of the elements concurring to the transversal response.

The seismic analysis took into account the church division into macroelements. In this case the concept of macroelement was applied to the transversal response. In this church, these elements seemed to be the most vulnerable to seismic action. In fact, the church has wide naves, slender walls and pillars, and the presence of elements of remarkable span makes this part of the structure particularly vulnerable.
Hence the vulnerability of this particular structural system, made up of the central nave arch and pillars, the lateral nave arches and the buttresses, was pointed out.

The *FEM* analysis was carried out through the general purpose *FEM* code *ANSYS rel. 5.7*. The *FEM* model was built up using large deflection plastic shells (SHELL 91 - *Nonlinear layered structural shell*). This element is defined by eight nodes, layer thicknesses, layer material direction angles, and orthotropic material properties; it has six degrees of freedom at each node. The *FEM*-model geometry is representative of the elements concurring to the transversal response of the church: a three-nave arch-and-pillars system. This model includes all the structural elements: transversal arches, pillars and counterforts, vaults, portion of the lateral walls corresponding to a bay (figure 10).

The vaults of the central and lateral naves were modelled considering half a bay for each part, and appropriate constraints were imposed to simulate symmetrical conditions. In the portion of the lateral walls considered, openings and appropriate constrains were taken into account. Different materials were used in the elements, in order to describe the structural behaviour in the best way.

A constitutive model capable of describing the non-linear behaviour of masonry structures, also for low stress levels, was chosen: in this case, the non-linear constitutive model, Podestà [13], implemented in the *FEM* code *ANSYS*. This constitutive model considers the limited tensile strength of fragile materials. The damage model allows us to simulate the progressive degradation of the stiffness (non-linear response) until the pre-chosen value of the failure stress and the consequent softening phase. Such a model does not consider, therefore, the cracks as localized phenomena, but it simulates them through an inelastic equivalent deformation field. The model, moreover, allows us to consider the shear failure with friction, adopting the Coulomb criterion. Besides the inelastic deformations, due to tensile stress (opening of the cracks), the sliding can also be simulated. Finally, the crushing is considered in those cases in which the wall section is strongly reduced.

In order to understand the dynamic properties of the global structure, a modal analysis was carried out and the first vibration period (T_0 = 0.81 s) and the first mode of vibration were evaluated.

A non-linear static incremental finite element simulation was carried out. The horizontal acceleration in unit of g versus the horizontal displacement of the node at the top of the central nave arch was plotted.

In Model 1, in correspondence of 0.14 g, the curve has not yet reached an asymptotic behaviour; that is to say, further strength sources are present (figure 11).

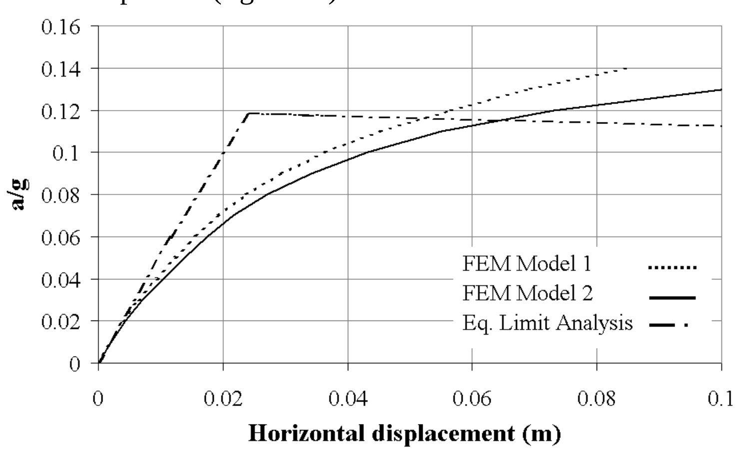

Figure 11. Horizontal acceleration (in unit of g) vs. horizontal displacement - Model 1, 2 and ELA.

Using a continuum model, the damage is not concentrated in a failure section, but a wider portion of masonry is subjected to this phenomenon. So the global behaviour of the structure does not show a sudden strength decrease. Moreover, in case of the severe damage level, the convergence problem of the solution implies a not clearly detectable asymptote.

Another non-linear static incremental finite element simulation has been performed, in order to evaluate the sensitivity to the presence of the vaults (Model 2). A lower value of the Young modulus has been assigned to the material of the vaults; through this parameter, these elements represent only a dead load on

the masonry transversal walls. In this case, the curve shows an asymptotic behaviour in correspondence to a lower value of the horizontal acceleration (figure 11). This fact reveals the structural and strengthening effect of the stone vaults in the church.
The inelastic maximum deformation in the resistant elements is displayed in figure 12.

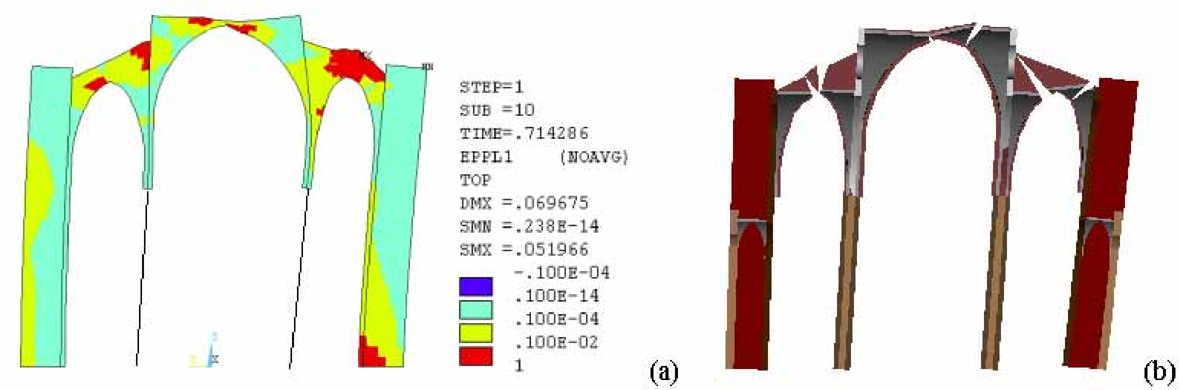

Figure 12. (a) Inelastic maximum deformation; (b) Suggested kinematism.

Analysing the structure, we can notice that the damage patterns suggest a particular kinematism. Through the Equilibrium Limit Analysis, we defined a capacity curve, in which the slope of the elastic branch is correlated to the first vibration period. A comparison between capacity curves by *FEM* and limit analysis was made (figure 11): in the *FEM* model the horizontal displacement of the top point was appropriately scaled in order to represent the horizontal displacement of a point at the mass centroid height. An acceptable correspondence may be noticed between *FEM* and limit analysis in the displacement range for which there is the connection of the elastic and the non-linear phase. In fact, we must consider that in the *FEM* curve this transition occurs continuously, instead of the sudden change obtained by limit analysis results.

FINAL REMARKS

The proposed methodology may be applied using differently detailed data. The results obtained may be helpful in the evaluation of the seismic risk of the monuments and in the definition of the mitigation strategy related to this topic.
The level 1 methodology provides a general estimation of the vulnerability of the monumental heritage in a specific seismic area; this method may be applied using poor data or after a quick survey.
The level 2 methodology, instead, furnishes results more useful in case of a damage prevention plan, because it allows us to estimate, even if approximately, the effectiveness of a seismic improvement strategy (applied in a systematic way).
The level 3 methodology is quite similar to a single monument seismic analysis, even if it is developed in order to get a set of results easily comparable to those of the other monuments (acquired instead through simplified models) and in order to create a damage scenario.

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REFERENCES

1. Lagomarsino, S., Podestà, S., 2003, “Seismic vulnerability of ancient churches. Part2: statistical analysis of surveyed data and methods for risk analysis”, (accepted for publication).
2. Heyman, J., 1982, “The masonry arch”, Ellis Horwood L.td, Chichester.
3. Gambarotta, L., Lagomarsino, S., 1997, “Damage models for the seismic response of brick masonry shear walls, Part II: the continuum model and its applications”, Earth. Engineering and Structural Dynamics, **26.**
4. Galasco, A., Lagomarsino, S., Penna, A., Resemini, S., 2004, ”Non-linear seismic analysis of masonry structures”, Proc. of the 13th World Conference of Earth. Eng., paper no. 843, 1-6 August 2004, Vancouver, Canada, (in press).
5. Doglioni, F., Moretti, A., Petrini, V., 1994, “The churches and the earthquake”. Trieste: Edizioni LINT (in Italian).
6. Lagomarsino, S., Podestà, S., 2003, “Seismic vulnerability of ancient churches. Part1: the damage assessment and the emergency planning”, Earthquake Spectra, (accepted for publication).
7. Giovinazzi, S., Lagomarsino, S., 2002, “A methodology for the vulnerability analysis of built-up areas”, Proc. International Conference on "Earthquake loss estimation and risk reduction", Bucharest, October 2002.
8. Sandi H., Floricel I., 1995, “Analysis of seismic risk affecting the existing building stock”. Proc. of the 10^{th} European Conference on earthquake Engineering; Vol.3, pp: 1105-1110.
9. Lagomarsino, S., Podestà, S., 2003, “Damage and vulnerability assessment of the churches after the Molise earthquake (2002)”, Earthquake Spectra, (accepted for publication).
10. Freeman, S.A., 1998, “Development and use of capacity spectrum method”, Proceeding Sixth U.S. National Conference on Earthquake Engineering, Earthquake Engineering Research Inst., Oakland California.
11. Lagomarsino S., Penna A., 2003, “A Non-linear Model for Pushover and Dynamic Analysis of Masonry Buildings”, Int. Conf. on Computational & Experimental Eng. and Sc. - Analytical and Experimental Methods in Earthquake Structural Engineering Symposium, Corfu.
12. Resemini, S., Lagomarsino, S., 2004, “Seismic vulnerability of masonry arch bridges”, Proceeding 11th National Conference: The Earthquake Engineering in Italy, Genoa, January 2004 (in Italian).
13. Podestà, S., “A damage model for the analysis of the seismic response of monumental buildings”, Journal of Earthquake Engineering, (in press).
14. Calderini, C., Lagomarsino, S., “A micromechanical model for masonry structures”, Proc. of the Sixth International Symposium on Computer Method in Structural Masonry (STRUMAS), 22nd - 24th September, 2003, Rome, Italy, (in press).
15. Irizarry, J., Podestà, S., Resemini, S., 2002, “Capacity curves of monumental-heritage elements: the Santa Maria del Mar Church in Barcelona”, Proc. International Conference on "Earthquake loss estimation and risk reduction", Bucharest, October 2002.