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SEISMIC ASSESSMENT OF HISTORICAL NATURAL STONE MASONRY BUILDINGS THROUGH NON-LINEAR ANALYSIS

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ABSTRACT :

Seismic assessment of historical buildings is a complex problem due to the wide variety of involved aspects, such as the quality of masonry, the structural systems, the large effort in inspection and diagnosis, the economical and cultural implications.
In the last years significant developments have occurred with respect to the possibilities of experimental and numerical analysis of ancient cultural heritage buildings. An example is the TREMURI program allows to obtain complete 3D macro-element models on which global non-linear static and dynamic analyses can be carried out, with limited computational efforts. By means of internal variables, the macro-element considers both the shear damage failure mode and its evolution, controlling the strength deterioration and the stiffness degradation, and rocking mechanisms, with toe crushing effect.
In this paper the results of the experimental investigations and non-linear analyses on a historical building are shown. The analysed building is the St. Michele Arcangelo Monastery in Gragnano (Naples, Italy), that is a large Campanian natural stone masonry building with an internal cloister. Analyses were carried out, through laboratory tests and in-situ investigations; they underline critical issues related to the seismic response of historical buildings, such as the variability of traditional material properties, the different construction techniques, the limited knowledge on previous damage or the limitations in inspections and tests due to conservation issues for buildings of historical value. Finally several interesting considerations about various strengthening configurations are reported, in order to assess the effects of some common interventions on historical masonry buildings, considering both the global response and local collapse modes.

KEYWORDS: masonry, assessment, non-linear analysis, cultural heritage

1. INTRODUCTION

Seismic assessment of historical masonry buildings is an evolving matter, whose importance has been highlighted in the last years in Italy with the publication of recent guidelines for seismic design of strengthening interventions. Despite the great effort carried out by the scientific community, nowadays a general modeling and analysis tool is not yet available, due to the wide variety of the aspects involved, including the quality of masonry, the structural system, the large effort in inspection and diagnosis, the economical and cultural implications.
The design approach for interventions on historical buildings does not require the complete seismic upgrading to a predefined seismic safety level, but it allows to reach only a partial upgrading in order to respect the preservation requirements accepting a level of seismic protection lower than the one prescribed for new structures. In this paper all phases suggested by “Guidelines for the evaluation and reduction of seismic risk of buildings of the architectural heritage” were retraced with respect to the seismic safety evaluation of the St. Michele Arcangelo Monastery in Gragnano (Italy).

2. ITALIAN GUIDELINES

On October 2007 Italian “Guidelines for the evaluation and reduction of seismic risk of buildings of the architectural heritage” were published. These guidelines introduced the concept of the seismic enhancement, to be intended as a partial upgrading able to improve the seismic performance of a historical building also respecting preservation requirements. Thus, seismic enhancement is different from seismic assessment that is the classic full seismic upgrading required by the technical guidelines for ordinary buildings.
The above mentioned Guidelines suggest an approach based on three phases:

- knowledge acquisition;
- seismic safety evaluation;
- structural intervention design.

The scope of this approach is to create a procedure based on an accurate knowledge of the structure that indicates an objective evaluation of the seismic safety level of the building and suggests the most convenient intervention. The three phases are briefly described in the following.

2.1. Knowledge of the building

The knowledge of a building implies geometrical mapping, experimental investigation and historical research. Generally geometrical mapping is easily carried out, while experimental investigations have to preserve the historical value of the building. In this case, historical researches can be very useful since they can help defining the evolutionary process of the building and they often give important information about the construction sequence.
Historical information about a building can also be used as a verification tool: once the most significant seismic events are individuated, it is possible to look for signs of their damages on the structure, in order to have indirect information about building capacity to resist to seismic actions and individuate the most critical damage mechanisms.
Hence, the final aim of this phase is to define a model that allows to give a qualitative interpretation of the structural behavior and subsequently to perform a structural analysis able to give a quantitative evaluation of the seismic safety.
Once the knowledge phase is completed, it is possible to define the confidence factor F_C that will be the material safety factor to be used for the seismic evaluation. This factor is calculated through the following equation:

$$F_C = 1 + \sum_{k=1}^{4} F_{Ck} \tag{4.1}$$

where the four terms are based on the elaboration level concerning:

- geometric survey;
- material survey and constructional details;
- mechanical properties;
- geotechnical soil and foundation.

In table 1.1.1 all possible values of these terms are reported.

Table 1.1.1 Values of the 4 terms necessary to define the confidence factor

Geometric survey	**Material survey and constructive details**	**Mechanical properties of material**	**Geotechnical soil and foundation**
complete geometric survey F_{C1}=0,05	limited survey of materials and constructive details F_{C2}=0,12	mechanical properties obtained by available data F_{C3}=0,12	limited investigations on geotechnical soil and foundation structure F_{C4}=0,06
complete geometric survey and graphic representation of cracks and deformations F_{C1}=0	extensive survey of materials and constructive details F_{C2}=0,06	limited investigations on mechanical properties F_{C3}=0,06	availability of geological and foundation structure data; limited investigations on soil and foundation F_{C3}=0,03
	exhaustive survey of materials and constructive details F_{C2}=0	extensive investigations on mechanical properties F_{C3}=0	extensive or exhaustive investigations on soil and foundation F_{C3}=0

The geometrical survey must be conducted with a level of detail coherent with the one utilized in the analytical model. If the geometrical survey includes also a description of cracks and deformations, F_{C1} can be assumed equal to 0.
The aim of the material survey (masonry typology, slab typology, vault structure, etc.) and constructive details identification (connections between walls, possible weaknesses, type of slabs and degree of connection with the walls, thrust reduction elements, material deterioration etc.) is to individuate all the constructive typologies of the building and their localization, paying particular attention to the aspects that can trigger local collapse mechanisms.
Regarding the definition of F_{C3}, it is important to underline that often different masonry typologies are used to realize the structure. In these cases it seems correct to correlate the F_{C3} factor to the masonry typology which is most relevant for the seismic analysis.
The definition of the F_{C4} factor depends on the influence that the foundation system can have on the collapse mechanisms: if the collapse mechanisms are assumed not to be influenced by the geotechnical parameters, it is possible to use F_{C4}=0. Otherwise the F_{C4} factor must be chosen depending on the type of investigations carried out.

2.2. The seismic safety evaluation

The guidelines introduce a new model for the evaluation of seismic safety through the definition of three levels of investigation:
LV1: territorial-scale simplified seismic evaluation;
LV2: seismic evaluation to be used in case of local interventions on a building;
LV3: deep evaluation of the seismic safety of a building.
LV1 allows evaluating the collapse acceleration of buildings by means of simplified models based on a limited number of geometrical and mechanical parameters or qualitative tools (visual test, construction features, and stratigraphic survey). LV2 has the aim to evaluate seismic safety when local interventions on single frames of a building are carried out. It is important to underline that LV2 can be used only when local interventions do not modify the structural behavior of the building. Otherwise it is necessary to use LV3. Such level is based on the use of models that simulate the global structural behaviour of the building and allow estimating the values of acceleration leading the structure to each limit state. These accelerations will be compared to the ones expected according to the Seismic Code.

The expected acceleration can be adjusted through a γ_I factor which depends on both strategic relevance and type of use of the building.

Table 2.2.1 Values of γ_I factor

Type of use of the building	**Strategic relevance of the building**		
	Limited	*Normal*	*High*
Occasional	0.50	0.65	0.80
Frequent	0.65	0.80	1.00
Very frequent	0.80	1.00	1.20

The seismic evaluation is based on the Seismic Safety Index (I_{SS}), obtained as the ratio between limit state acceleration and expected acceleration for the analyzed building.

$$I_{SS} = \frac{a_{LS}}{\gamma_I Sa_{LS,\exp}} \tag{4.1}$$

I_{SS} values larger than 1 indicate that the analyzed building is able to resist the expected seismic action; $I_{SS}<1$ means that the level of seismic safety of the building is lower than the required one. It is useful to underline that the I_{SS} check is not mandatory, but it represents an important quantitative parameter to consider in order to express a final qualitative evaluation in which other important involved aspects (conservation and preservation requirements, safety demand, strategic relevance and type of use of the building) are considered.
In other words, it is possible to accept values of I_{SS} smaller than 1 if it is demonstrated that interventions needed to fully satisfy structural checks are in conflict with preservation requirements.
Anyway, when strategic or relevant activities are carried out in the analyzed building (hospital, school, fire or police station, etc.), the I_{SS} verification, even if not mandatory, assumes a relevant importance. In these cases, it could be preferable to dislocate such activities to other buildings, in order to avoid that preservation requirements could engrave on the correct working of important strategic activities or on consequences of a potential collapse.

2.3. Upgrading interventions

Structural interventions, aiming at seismic vulnerability reduction, have as their main objective the preservation of materials and original resistance structural mechanism, as long as it does not cause early collapse of the building.
Moreover their choice must depend on results of the evaluation phase. In particular, interventions will have to reach the safety and durability of the building producing the minimum impact on it and respecting, if possible, both the original structural configuration and all subsequent modifications.
From this point of view, damaged structural elements must be repaired as long as possible while it should be avoided element substitution and utilization of innovative systems, unless their compatibility with original materials was demonstrated.
Finally, particular attention should be given to executive phase of interventions in order to verify their effectiveness and avoid damages that could make worse mechanical properties of masonry or framework structural mechanisms.

3. SEISMIC MODELING AND ANALYSIS

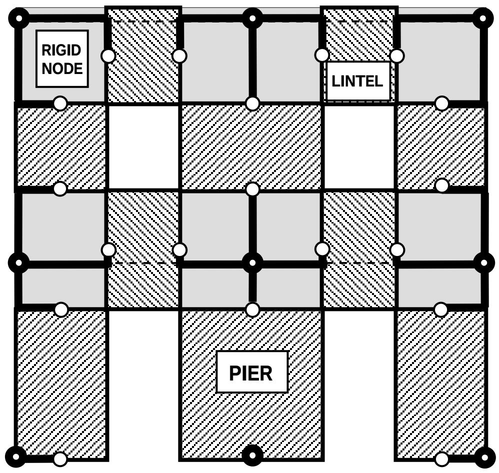
3.1. Global non-linear seismic analysis

The main source of vulnerability of existing masonry buildings is certainly associated to local failure modes, usually involving the out-of-plane overturning of façades or of portions of the external walls. The vulnerability to local failure mechanisms is mainly due to lack of connection between orthogonal walls and between walls and floors. Proper connection devices (e.g. tie-rods) can increase the seismic safety with respect to local damages and they allow the building to behave as an entire structure with a seismic response governed by the

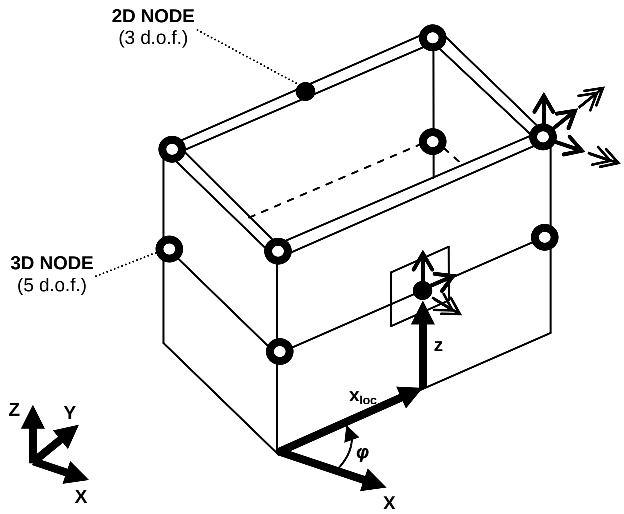
in-plane behaviour of walls and horizontal structures (floors, vaults and roofs).
The seismic response of heritage buildings, usually characterised by significant structural irregularities, presents a highly nonlinear behaviour, even for relatively low levels of lateral deformation. For this reason linear analyses are not an appropriate tool for the assessment of seismic behaviour of such structures. On the other hand, nonlinear static and dynamic analyses allow to take into account the different level of structural knowledge and follow the damage evolution and distribution among structural elements.

3.2. The TREMURI model

The 3-dimensional modelling of whole unreinforced masonry URM buildings starts from the identification of walls and floors as bearing structure, both referring to vertical and horizontal loads. The local flexural behaviour of the floors and the wall out-of-plane response are not computed because they are considered negligible with respect to the global building response, which is governed by their in-plane behaviour (a global seismic response is possible only if vertical and horizontal elements are properly connected). The wall is modelled as a frame of non-linear elements, which constitutive relationship is formulated to approximate the actual damage behaviour of masonry panels. The numerical models and analysis procedures, described in the rest, have been incorporated into the TREMURI program [Galasco et al., 2007].
A frame-type representation of the in-plane behaviour of masonry walls is adopted: each wall of the building is subdivided into piers and lintels, modelled by non-linear macro-elements, connected by rigid areas (nodes). The presence of stringcourses (beam elements), tie-rods (non-compressive rod elements), previous damage, heterogeneous masonry portions, gaps and irregularities can be easily included in the structural model.
The non-linear macro-element model, representative of a whole masonry panel, is adopted for the 2-nodes elements representing piers and lintels. Rigid end offsets are used to transfer static and kinematic variables between element ends and nodes.



(a)



(b)

Figure 3.2. Macro-element modelling of a masonry wall (a); 3-D building model assembling (b).

A global Cartesian coordinate system (X,Y,Z) is defined and the wall vertical planes are identified by the coordinates of one point and the angle formed with X axis. In this way, the walls can be modelled as planar frames in the local coordinate system and internal nodes can still be 2-dimensional nodes with 3 d.o.f.. Floor elements, modelled as orthotropic membrane finite elements, with 3 or 4 nodes, are identified by a principal direction, with Young modulus E_1, while E_2 is the Young modulus along the perpendicular direction, ν is the Poisson ratio and $G_{1,2}$ the shear modulus. $G_{1,2}$ represents the in-plane floor shear stiffness which governs the horizontal actions repartition between different walls.
The macro-element adopted in this work is a two-nodes bilinear elastic perfectly plastic model which incorporates the shear and flexure strength criteria suggested in the Italian Code (NTC08) and Eurocode 6 (EC6).

4. THE CASE OF THE ST. MICHELE ARCANGELO MONASTERY IN GRAGNANO

4.1. Description

St. Michele Arcangelo Monastery is a large Campanian natural stone masonry building that was built at the beginning of XIV century, in Gragnano, a little town near Naples (Italy). In 1861, after Italy's unification, it was suppressed. Later it was destined to be a school, and it was definitely deserted after Irpinia earthquake in 1980. Currently, a great part of this building is neglected and only a small part is still in use.
The building has the typical architectural system of a monastery; it is composed of two levels and has a wide square cloister (about 30m).
At the ground floor the cloister is delimited by a refined colonnade realized with ribbed vaults that lean on square clay bricks masonry columns. Upstairs there are four corridors and numerous rooms, that were originally destined to be nuns bedrooms.
Compactness of the building is interrupted by two large rooms that were originally used as storage. Floor diaphragm is realized through masonry vaults except some areas in which original vaults were substituted with floor realized with steel I-beams and clay tiles.

Figure 1: St. Michele Arcangelo Monastery in Gragnano

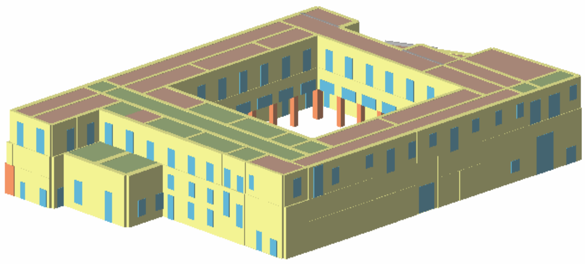


Figure 2: TREMURI model

4.2. Experimental investigation

All experimental investigations have been carried out according to Guidelines in order to have a deep knowledge of the building. The first phase consists of a geometrical and structural survey of the whole building (F_{C1}=0.05) and material and cracks pattern investigation (F_{C2}=0.00). Two different masonry typologies were identified. The first was realized with grey tuff balk units and was utilized for the most of building walls; while the second typology was a clay bricks masonry, used for all columns and arches at the ground floor in the area near the cloister.
In order to evaluate material mechanical properties, double flat jack experimental tests were carried out (F_{C3}=0.00). Experimental results are reported in the table 4.2.1. Regarding geological and foundation structure data limited investigations on foundation structure were carried out (F_{C4}=0.06).
Finally the confidence factor F_C obtained is 1,11.

Table 4.2.1 Experimental results of double flat jack tests

	Cracking strength [MPa]	Strength [MPa]	Young Modulus [MPa]
Masonry of grey tuff	1,44	1,60	770,50
Masonry of brick and mortar	3,39	>3,48	2285,40

4.3. Mechanical model and analysis

Building seismic analysis has been carried out because of the need to use it as Gragnano City Hall and Museum. Mechanical parameters of masonry used in the analysis (derived from the current code suggestions) are reported in Table 4.3.1.

Table 4.3.1 Mechanical properties of masonry typologies

Masonry typologies	f_m **[MPa]**	τ_0 **[MPa]**	***E* [MPa]**	***G* [MPa]**	***W* [kN/m^3]**
Grey tuff masonry	1,35	0,047	770	270	13
Clay brick masonry	3,10	0,103	2285	525	18

In order to carry out analysis according to Italian Seismic Code, in situ expected maximum horizontal acceleration has been calculated considering geographic coordinates. The obtained value has been multiplied by γ_I =1,2 factor to consider strategic relevance of the building (high) and its expected use (very frequent). In this way a seismic event was considered having an excess probability, in the period of 50 years, of 6,5% for severe damage limit state (SDLS) and 40% for limited damage limit state (LDLS).
Moreover a S=2,16 factor was introduced in order to take into account the effects of the topographic configuration on the seismic behaviour of the monastery.
Building structural behavior analysis has been carried out in 3 phases: the first one concerned analysis on original building conditions. This analysis, identified by SF-1 code, has allowed highlighting structure mechanical behavior in case of earthquake. Moreover analysis carried out in this phase allowed individuating all local collapse mechanisms and understanding that the main predictable local mechanisms were out of plane overturning mechanisms for walls on north and east sides.
In the second phase installation of tie rods has been provided in order to avoid the activation of the local mechanisms that had caused the premature failure of the structure in the SF-1 analysis. At the end of this phase a new structural model has been created in which several structural elements were designed in order to avoid local collapse mechanisms. This new model, identified with SP-1 code, has highlighted the good effect of interventions: local mechanism activation was avoided and seismic safety index related to SDLS has passed from 0,701 to 1,577 with an increase of over 100%. This proves that provided interventions, blocking the activation of the local mechanisms, had a beneficial effect also on the whole building behavior.
The third phase of the study aimed to carry out analysis to evaluate seismic behavior of the monastery after interventions. Due to building degradation, the substitution of the existing covering floor with reinforced concrete riddles and a wood floor having a good connectivity to the wall was provided.
Good mechanical behavior highlighted by SP-1 analysis suggested to assure that provided interventions will not modify monastery original functioning. For this reason, according to “Guidelines for the evaluation and reduction of seismic risk of buildings of the architectural heritage”, it has been foreseen a new floor having a mechanical behavior very close to the oldest one. Analysis concerning this phase has been identified with SP-2 code.
In Table 4.3.2. results of the three analyses are reported. Particularly, information about local mechanisms activation, expected peak ground accelerations at the base of the structure related to both SDLS and LDLS are reported. Moreover accelerations that cause the attainment of both SDLS and LDLS and related seismic safety indexes are reported.

Table 4.3.2. Results of analyses

ID	**Expected accelerations**		**Local mechanisms activation**	**Seismic assessment**			
	$a_{SDLS\text{-}exp}$ **[m/s^2]**	$a_{LDLS\text{-}exp}$ **[m/s^2]**		a_{SDLS} **[m/s^2]**	a_{LDLS} **[m/s^2]**	$I_{SS,SDLS}$ **[-]**	$I_{SS,LDLS}$ **[-]**
SF-1	2,90	1,485	YES	2,034	/	0,701	/
SP-1			NO	4,575	4,697	1,577	3,162
SP-2			NO	4,575	4,782	1,577	3,220

From data reported in table 4.3.2., it is possible to notice that interventions with tie rods, as well as have prevented local mechanisms activation, have considerably improved seismic safety index at severe damage limit state, while substitution of the covering floor did not change mechanical global behavior of the monastery. In fact it is possible to observe that passing from original configuration (SF-1) to the intermediate configuration (SP-1), there is a noticeable increase of $I_{SS,SDLS}$ index, while passing from intermediate configuration (SP-1) to the final configuration (SP-2) seismic safety indexes have a little improvement, due to the realization of reinforced concrete riddles near covering floor.

5. CONCLUSION

Seismic vulnerability analysis of the monastery, executed through non linear static analyses, allowed retracing all phases suggested by "Guidelines for the evaluation and reduction of seismic risk of buildings of the architectural heritage" and highlighting the importance of investigation phases that guarantee a deep knowledge of the building and help to create correct structural models.
Monastery analysis, organized in three phases, allowed to evaluate the global level effectiveness of interventions realized to avoid collapse local mechanisms. From data reported in table 4.3.2., it is possible to notice that a considerable improvement of the seismic safety index at SDLS was achieved due to the introduction of steel tie rods and reinforced concrete riddles.

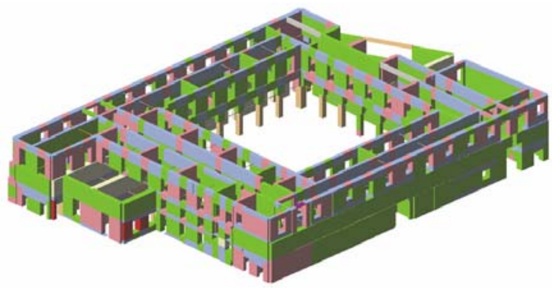


Figure 5.1: collapse step related to SP-1 analysis

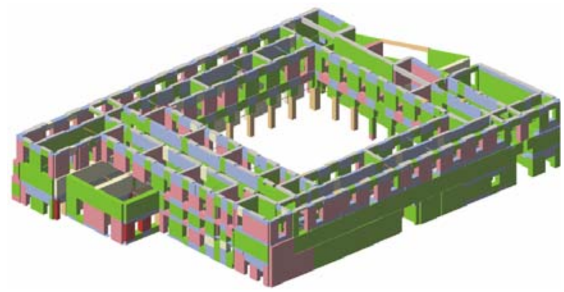


Figure 5.2: collapse step related to SP-2 analysis

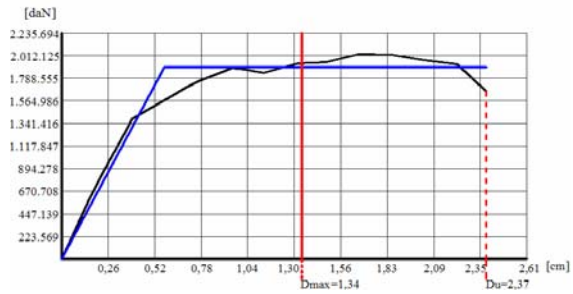


Figure 5.3: Pushover curve of SP-1

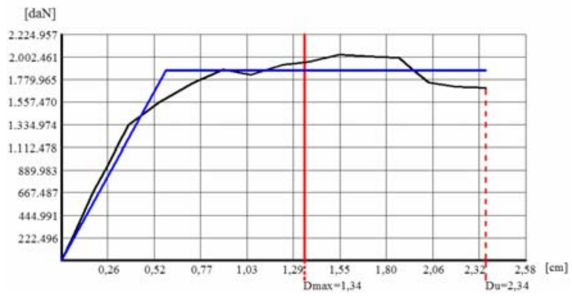


Figure 5.4: Pushover curve of SP-2

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