EXPERIMENTAL SEISMIC ANALYSIS OF MONUMENTS FOR RISK MITIGATION

L. Cavaleri[1], F. Cristina[2] and G. Zingone[2]

[1] *Professor, Dipartimento di Ingegneria Strutturale e Geotecnica,University of Palermo. Italy*
[2] *Members of Centro di Prevenzione e Istruzione Sismica,CE.P.I.S., Palermo. Italy*
Email: cavaleri@diseg.unipa.it

ABSTRACT :

An experimental investigation, at the moment in progress, for the evaluation of the seismic vulnerability of a monumental structure and the effects of some reinforcement devices are presented. Referring to this monumental structure, that is a church, the authors obtained the dynamically identified analytical model and recognized that the drum-dome system was one of the macro-elements with highest risk. This result was obtained by analyzing the linear behaviour. In order to study the nonlinear behaviour, tests on reduced scale models were necessary. The models, with and without risk mitigation reinforcements, were subjected to seismic input with rising intensity in order to know the effects of the reinforcements themselves. A study was also begun in order to determine the ultimate load and ductility resources given by the various types of reinforcements used. Through the paper some details of the above investigation are discussed and the last results are presented.

KEYWORDS: masonry, churches, drum-dome systems, modelling, retrofitting.

1. INTRODUCTION

The experiences and the results obtained while carrying out the research project on seismic risk prevention for the cultural patrimony of the Catania area (Catania Project 2) allowed to show the usefulness of employing structural analysis methodologies based on techniques of dynamic identification.

Specifically, in relation to the churches with one or more aisles having dome and drum, very widespread in the whole Mediterranean area, the authors tried to develop an analysis procedure in the linear field, which, by using a dynamically identified model, made it possible to identify the macro-elements characterized by the greatest risk. Among them the dome-drum subsystem was recognized. Referring to this subsystem, the analysis has been taken deeper in order to appraise the behaviour in the nonlinear state.

Referring to this second part of the investigation, as it was not possible to do tests in situ, it became necessary to resort to dynamic tests on large-scale models. The responses obtained submitting the model to input with increasing intensity proved particularly useful for getting information on the improvement capacities of different types of reinforcement, on seismic behaviour in the cracked state, on the evolution of the damage and on the ductility resources offered by the system in the different situations.

Simultaneously some numerical simulations were carried out by using three-dimensional F.E. models and adopting proper criteria in order to know the ductility resources of the system with the types of reinforcements used.

2. LINEAR SEISMIC ANALISYS

In this section a synthesis is presented of the investigation that allowed the recognizing of the elements of the churches above mentioned characterized by the highest seismic risk. Much more details are discussed in (Valente and Zingone, 2004; Zingone and Valente, 2005; Zingone et al., 2006a; Zingone et al., 2006b). The investigation was characterized by different steps, the first of which was the choice of a sample church, representative of this kind of structures.

2.1. Choice of the sample monument to be studied

For the choice of the church to be studied, a preliminary investigation extended to the whole historic area of Catania was carried out, consisting of collecting data about the structure of a suitable number of churches. By using the above data two indexes were calculated in agreement to Zingone et al. (1999). The first one, Si, defined safety index, is linked to the typological characteristics. The second one, Di, defined damage index, is representative of the state of damage. For each of the examined churches the 1st and 2nd level of vulnerability were calculated depending on the above indexes.

On the basis of the values obtained, a classification in terms of vulnerability was done that allowed the choice of the sample church. In details the one characterized by the greatest vulnerability levels was chosen, that is the San Nicolò l'Arena church.

2.2. Definition of the dynamically identified model

The plan and the section of the examined system are represented in Fig. 1; in the plan the part of the structure taken as the basis of the analysis is bounded by a dashed line rectangle. The finite elements model taken as the basis of the analysis, including part of the foundation soil to take the interaction with the structure into account, is inserted in Fig. 2.

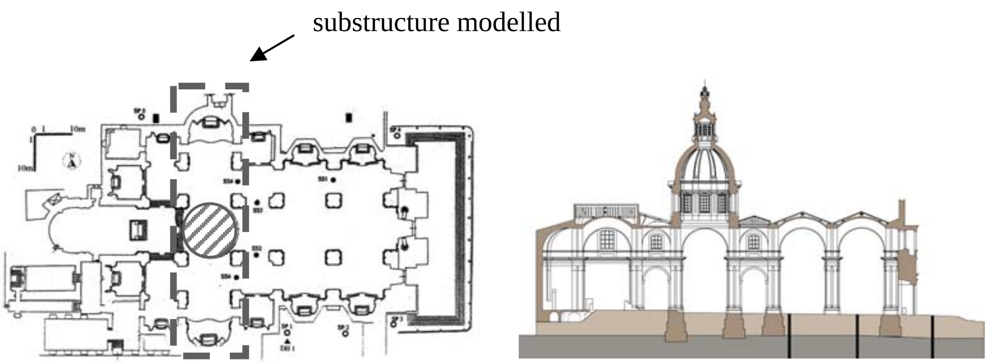


Figure 1 Plan and section of San Nicolò l'Arena church

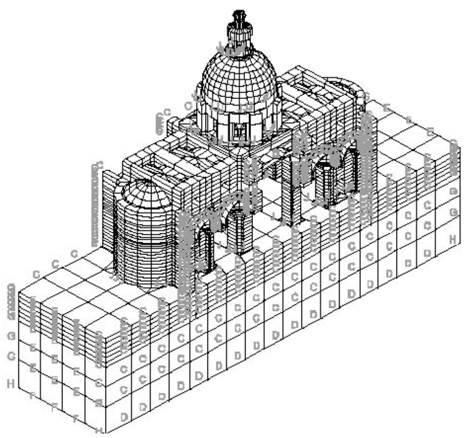
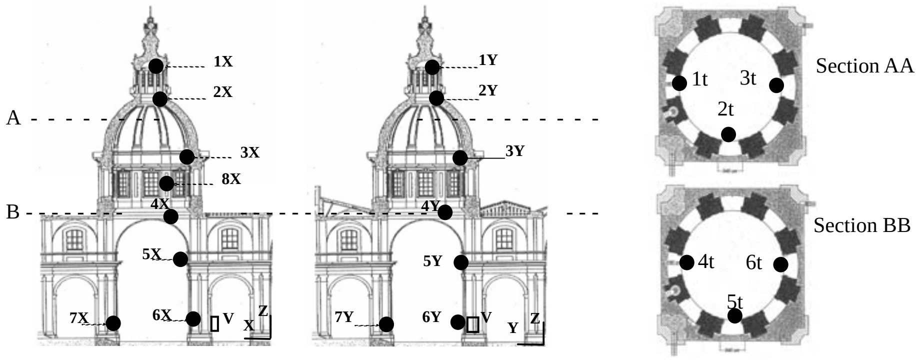


Figure 2 F.E.M. of the substructure and of its foundation soil

In the first model formulation the following elastic moduli were assumed: E=4000 MPa for the structure and E=2000 MPa for the foundation. These moduli were updated on the basis of a dynamic identification after vibration test in situ carried out by exciting the system by a vibrodyne applied at the base of one of the columns.

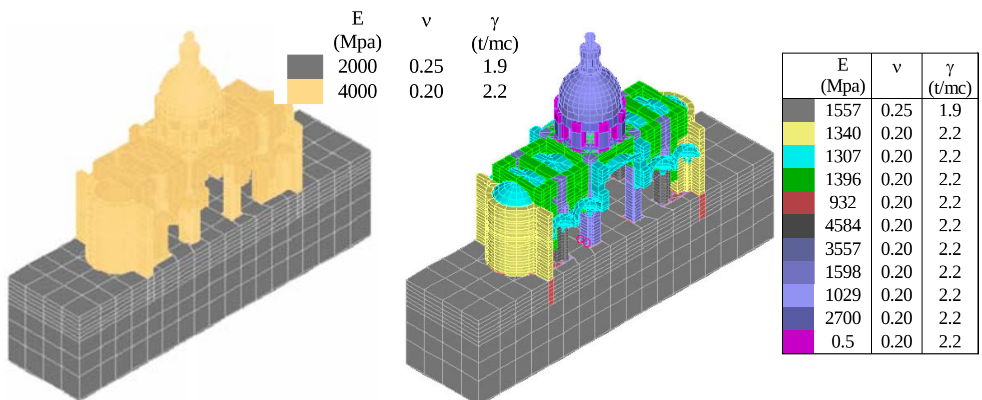


a) b)

Figure 3 Measurement network for detecting lateral (a) and torsional (b) response

Fig. 3a shows the position of the vibrodyne (V) together with the location of the accelerometers (•) along x and y directions, for the recordering of the lateral response. The measurement devices, displaced as in Fig. 3b, had the aim to measure also the torsional response.

The identified model was obtained by an iterative process of calibration (Zingone and Valente, 2005). In Fig 4 one can see the model before the calibration characterized by two elastic moduli (corresponding to two colours) and the model after the calibration characterized by a different distribution of the elastic moduli corresponding to different colors (Fig. 4b).



a) b)

Figure 4 Model before (a) and after (b) the calibration

2.3. Macro-elements under risk

The analysis of the signals in terms of accelerations revealed the main model shapes and the corresponding frequencies by means of that the model in Fig. 4a was calibrated. Once the model in Fig. 4b was obtained, as a result of the calibration, a seismic analysis was carried out that evidenced the vulnerability of the dome-drum system. In Fig. 5 two of the modal shapes of the substructure studied are shown.

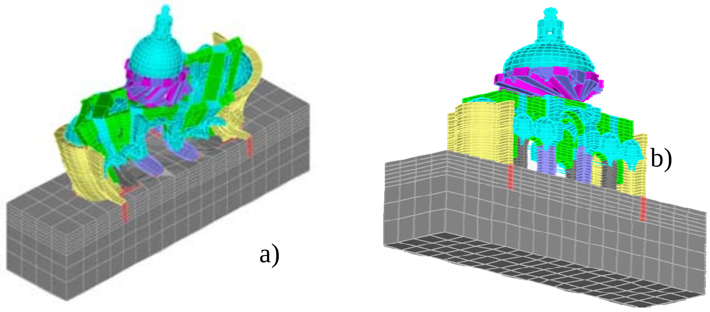


Figure 5 Two vibration mode obtained by the identified model

3. NONLINEAR SEISMIC ANALYSIS

In order to investigate the seismic behaviour of the dome-drum system in the nonlinear field, a 1:6 scale model was made using ashlars and calcarenite mortar with similar characteristics to the real ones. In this section some aspects of this analysis are discussed but much more detailed are in (Zingone et al., 2006b).

The model was submitted to seismic input using the vibrating table of the ENEA Experimental Centre in Casaccia. The three following configurations were analyzed: a) model in the absence of reinforcement -Fig. 6a; b) model strengthened with frames -Fig. 6b; c) model strengthened with braced frames (Fig. 6c). The reinforcing frames were made of steel profiles with a section of 70x50x4 mm, while the bracing elements were constituted of flat steel profiles with a section of 18x0.8 mm.

The models were submitted to time histories of seismic accelerations recorded in Italy, appropriately scaled, with an increase in the maximum acceleration peak at each following test. Each test was interrupted as soon as a reduction in the accelerations was recorded at the top of the model corresponding to the appearing of cracks at the head and foot of the masonry shear walls of the drum.

Figure 6 Model in the absence of reinforcement (a), model strengthened with frames (b) and with braced frames (c).

The models (b) and (c) were submitted to a history of horizontal accelerations, recorded at Colfiorito (Marche-Umbria), appropriately scaled, with an increase in the maximum peak acceleration in each test. The increasing of the peak acceleration was interrupted as soon as an abrupt reduction in the dominant frequencies of the system was recorded. Also in these two cases, cracks were detected involving the masonry rings placed above and below the delimitation planes of the masonry shear walls of the drum (Fig. 7).

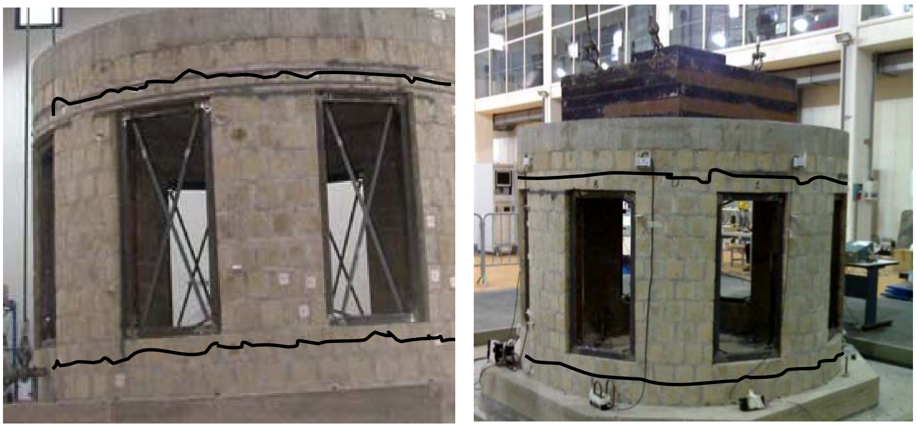


Figure 7 Location of the cracks for the model reinforced with frames and braced frames

The base accelerations were applied in two orthogonal directions. In Fig. 8 the maximum peak accelerations at the top of the model in the three different configurations varying the peak ground acceleration (PGA), along one of the two orthogonal direction considered, are depicted proving the beneficial effect of the reinforcing in terms of stiffness and strength.

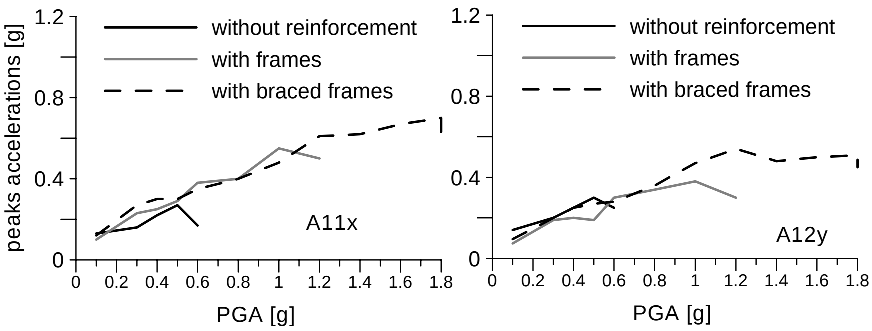


Figure 8 Peaks of response in terms of acceleration in two measure points on the top of the models

Each branch of the curves in Fig. 8 shows that to an increase in PGA there generally corresponds an increase in the acceleration peak at every measurement point, but there is a value of the PGA which correspond to a decrease of the peaks acceleration associated because of a reduction in stiffness, recorded even before the damage was clearly manifested.

By Fig. 8 the positive effect of the reinforcement is evidenced. Clearly Fig. 8 highlighs the fact that the reinforcements give stiffness and strength: this deriving from the possibility of increasing of the maximum peak accelerations at the top once the reinforcement is applied.

4. APPROXIMATED EVALUATION OF THE DUCTILITY

There now follows a numerical analysis of the cases taken as the basis of the experimental investigation for the purpose of deducing the ductility of the system with and without reinforcement.

4.1 Definition of the computational model

The dome-drum macro-element was modelled as shown in Fig. 9 (8-node solid 3D of SAP Nonlinear was used). The model included 1748 nodes and 704 elements.

The base of the drum was constrained at the base in all 6 degrees of freedom. The top plate was considered to have rigid behaviour for taking the presence of the crowning ashlar into account. On the top there were considered to act a horizontal load F as a static seismic action and a vertical load of about 90 kN uniformly distributed. The horizontal load was increased up to the limit stress for the materials was reached.

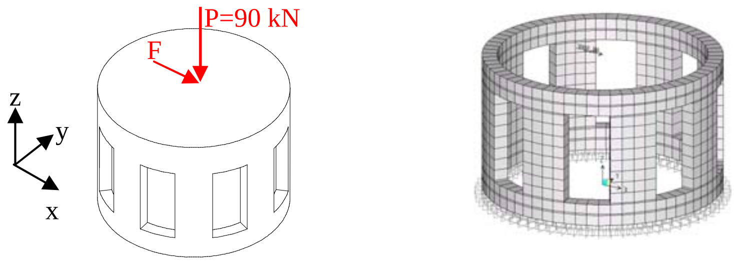


Figure 9 Model of the dome-drum macro-element.

4.2 Model in the absence of reinforcements

A linear analysis was carried out of the system both in the undamaged state and in the damaged state taking the collapse mechanisms experimentally observed into account.

A homogenized elastic modulus $E_{O,mur}$ was assumed equal to 1000 MPa, deduced through experimental tests on specimens of the masonry used, specific weight 16 kN/m³ and Poisson ratio 0.2. Fig. 10a shows qualitatively the stress state. The Figure highlights that the maximum stresses are concentrated in the sections at the head and base of the masonry walls limiting the windows. Once the maximum force and the corresponding displacement to the top was recorded the model was modified to take into account the state of damage produced during the experimental tests. In details a thin layer of finite elements simulating a material with low elastic modulus (approximately 1/10 of the elastic modulus of the undamaged material) was inserted (Fig. 10b).

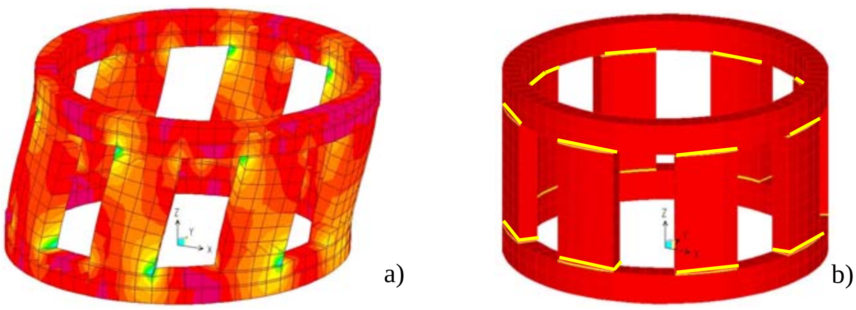


Figure 10 Stress/ strain state (a); model of the system in the damaged state (b).

In this condition the horizontal displacement at the top given by the same value of force producing the limit stress in the undamaged model was recorded and compared to the above displacement for the evaluation of the ductility.

4.3. Model reinforced with frames

The analysis process described before was repeated when frames were used as reinforcement. The models in the undamaged and in the damaged state are inserted in Fig 11. The model in the damaged state was defined taking the distribution of the cracks experimentally observed. In this case a thin layer of finite elements above and below the masonry shear walls was inserted in the damaged state characterized by a reduced elastic modulus (Fig. 11b). Also in this case the static horizontal force corresponding to the limit stress in the material was recorded. Further the corresponding displacement was observed referring to the undamaged model. Then, by using the same force before obtained, the horizontal displacement in the damaged model was recorded.

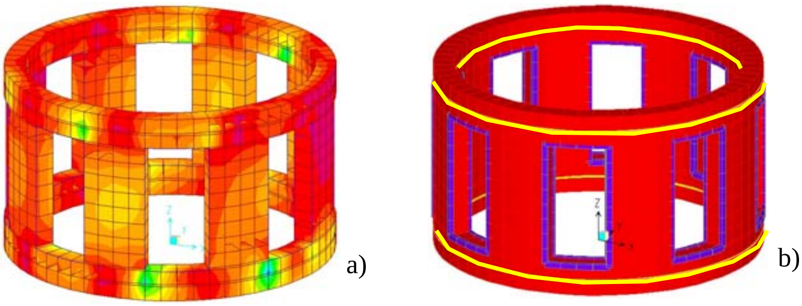


Figure 11 Stress state (a); model of the system in the damaged state (b)

The location of the maximum stress given by the model in the undamaged state resulted similar to what observed experimentally confirming the validity of the model used.

4.4 Model reinforced with braced frames

The analysis was repeated to obtain the response of the system reinforced with braced frames. In this case the state of stress and the model used are inserted in Fig. 12.

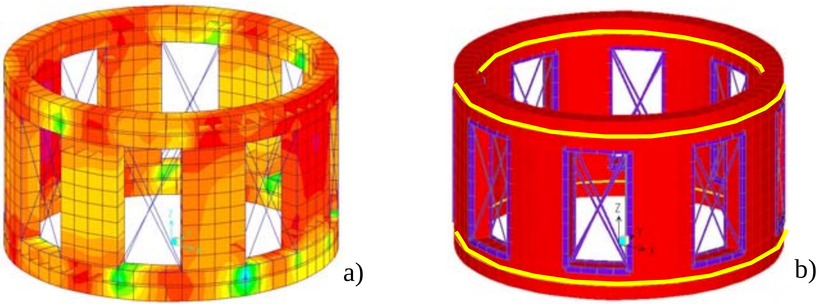


Figure 12 Stress state (a), model of the system in the damaged state (b).

5. MAIN RESULTS OF THE ANALYTICAL INVESTIGATION AND CONCLUSIONS

The most significant results obtained from the three configurations analyzed are summarized in Fig.13 in which the horizontal strength is related to the normalized horizontal top displacement $\delta/\delta e$, δ being the generic horizontal top displacement, corresponding to an assigned level of force, and δe is the displacement corresponding to the limit of strength of the material obtained from the undamaged model.

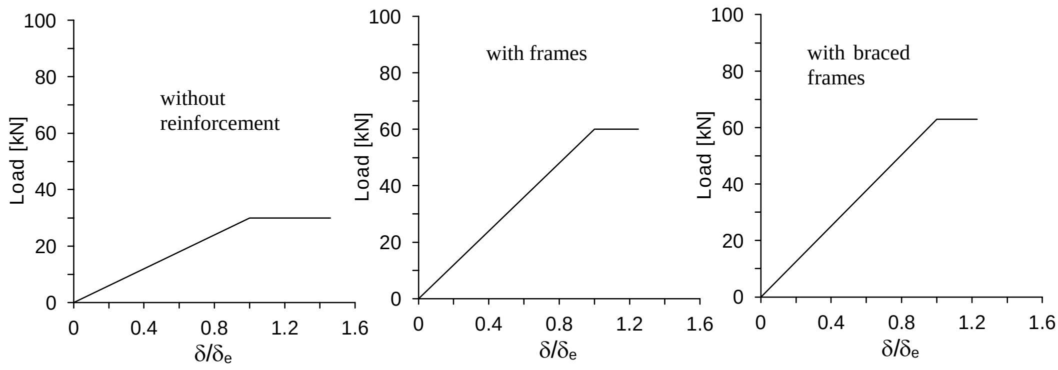


Figure 13 Horizontal load versus horizontal normalized displacement

Fig. 13 shows that reinforcements can condition the structural ductility in the sense of a reduction. Observe the reduction of ductility because of the steel frames. Further observe that the braced frames do not produce a significant increasing of strength too. This is due to the fact that the collapse mechanism changes involving the masonry up and down the windows where the effect of the reinforcement vanishes. This fact is evidenced also by the experimental results (Fig. 8) that show a minor increment of the peak accelerations when braced frames are used instead of frames. While the increment of peak accelerations, because of the inserting of the frames in comparison with the bare structure, evidences the positive effects of the frames in terms of strength.

The results clearly condition the choice of the reinforcement and suggest different questions on the proper reinforcements to use, able to increase the strength and not to reduce so much the ductility.

Actually a new type of reinforcement, including a damper, is under test with the aim to reduce the requesting of ductility, the results will be presented subsequently.

REFERENCES

Valente, G., Zingone, G. (2004). Identificazione dinamica dei sistemi strutturali a tipologia specialistica. *XI Convegno Nazionale ANIDIS "L'Ingegneria Sismica in Italia*. Genova, Italia.

Zingone G., Valente G. (2005). Methodology and techniques for seismic protection of the monumental patrimony. In *Seismic Prevention of Damage*, WIT Press.

Zingone, G., Cavaleri, L. Cucchiara, C. (2006b). L'impiego di tecniche di identificazione dinamica per la prevenzione e mitigazione del rischio da crolli. *Convegno Nazionale CRASH 06*. Messina, Italia.

Zingone, G., Cavaleri, L., Cucchiara, C., Tommaselli, A. (1999). Valutazione della vulnerabilità globale delle chiese in muratura attraverso l'analisi di macroelementi significativi. *IX Convegno Nazionale ANIDIS "L'Ingegneria Sismica in Italia"*. Torino, Italia.

Zingone, G., De Canio, G., Cavaleri, L. (2006a). On the improvement of monumental structure safety: a case study. *III ECCM*. Lisbona, Portogallo.