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DAMAGE IDENTIFICATION OF MONUMENTAL MASONRY STRUCTURES: THE CASE OF FOSSANOVA GOTHIC CHURCH

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Abstract. *In the present paper, the seismic behavior of a physical 1:5.5 scaled model of the church of the Fossanova Abbey is investigated by means of numerical and experimental analyses. Aiming at defining the seismic vulnerability of such a structural typology a wide experimental campaign was carried out. The achieved experimental results lead to the definition of a refined FE model reproducing the dynamic behavior of the whole structural complex. Then, the central transversal three-central bays of the church, as it mostly influences the seismic vulnerability of the Abbey, was investigated in a more detail by means of a shaking table test on a 1:5.5 scaled physical model in the Laboratory of the Institute for Earthquake Engineering and Engineering Seismology in Skopje. In the present paper a brief review of the numerical activity related to the prediction of the shaking table test response of the model is first proposed. Then, the identification of frequency decay during collapse is performed through decomposition of the measured power spectral density matrix. Finally, the localization and evolution of damage in the structure is analyzed. The obtained results shown that a very good agreement is achieved between the experimental data and the predictive/interpretative numerical analyses.*

1 INTRODUCTION

Gothic architecture spread as from the 12th Century and broke out during the Middle Ages in the cultural and religious area of the Christianity of Western Europe, with some trespasses in the Middle East and in the Slavic-Byzantine Europe. Many important abbeys were built in those areas, providing a key impulse to the regional economy and contributing to a general social, economic and cultural development. The most interested areas sprawl from the northern Countries (England) to those facing the Mediterranean Basin (Italy), but also spread out from the Western (Portugal) to the Eastern Countries, as Poland and Hungary. Monastic orders and in particular the Cistercian one, with its monasteries, had an important role for broaden the new architectonic message, adapting to the local traditions the technical and formal heritage received by the Gothic style [1], [2] and [3] .

Gothic cathedrals may result particularly sensitive to earthquake loading. Therefore, within the European research project “Earthquake Protection of Historical Buildings by Reversible Mixed Technologies” (PROHITECH), this structural typology has been investigated by means of shaking table tests on large scale models [4]. Based on a preliminary study devoted to define typological schemes and geometry which could be assumed as representative of many cases largely present in the seismic prone Mediterranean Countries, the Fossanova cathedral, which belongs to the Cistercian abbatial complex built in a small village in the central part of Italy, close to the city of Priverno (LT), has been selected as an interesting and reference example of pre-Gothic style church [5]. In order to assess the vulnerability of the church against seismic actions a wide numerical and experimental activity was developed. Firstly, the identification of the geometry of the main constructional parts as well as of the mechanical features of the constituting materials of the cathedral was carried out. Then, Ambient Vibration Tests were performed in order to characterize the dynamic behavior of the church and to calibrate refined FE models developed by the ABAQUS code [6]. At this purpose elastic FEM analyses were performed to predict the behavior of the three-central bays of the church, which were detected as the key-part of the structural complex [7], [8]. The recognized resistant unit about transversal direction was designed in length scale 1:5.5 according to "true replica" modeling principles and tested on the shaking table in the IZIIS Laboratory in Skopje [9]. The physical model was tested and the as-built model was loaded until heavy damage occurred. The structural response of the tested physical model has been deeply investigated by means of non-linear numerical analyses that has shown good agreements with experimental measurements [10].

In this paper, the identification of frequency decay during collapse is performed through decomposition of the measured power spectral density matrix. Finally, the localization and evolution of damage in the structure is analyzed. The obtained results has shown that a very good agreement is achieved between the experimental data and the predictive/interpretative numerical analyses.

2 THE FOSSANOVA ABBEY: MODEL AND EXPERIMENTAL TEST

The Fossanova Abbey (Fig. 1) was built in the XII century and opened in 1208. The architectural complex presents three rectangular aisles with seven bays, a transept and a rectangular apse. Between the main bay and the transept raises the skylight turret with a bell tower. The main dimensions are 69.85 m (length), 20.05 m (height), and 23.20 m (width). The nave, the aisles, the transept and the apse are covered by ogival cross vaults. Detailed information on the main dimensions of the bays are provided in De Matteis et al. [6].

Figure 1. The Fossanova church.

The previously mentioned vaulted system does not present ribs, but only ogival arches transversally oriented respect to the span and ogival arches placed on the confining walls (Fig. 2). The ridge-poles of the covering wood structure is supported by masonry columns placed on the boss of the transversal arches of the nave and apse. The crossing between the main bay and the transept is covered by a wide ogival cross vault with diagonal ribs sustained by four cross shaped columns delimiting a span with the dimensions of 9.15x8.85m.

Figure 2. The vaulted system of the Fossanova church.

The main structural elements constituting the central nave and the aisles are four longitudinal walls (west-east direction). The walls delimiting the nave are sustained by seven couples of cross-shaped piers (with dimensions of 1.80x1.80 m) with small columns laying on them and linked to the arches. The bays are delimited inside the church by columns with adjacent elements having a capital at the top. The columns-capital system supports the transversal arches of the nave. The external of the clearstory walls are delimited by the presence of buttresses with a hat on the top that reaches the height of 17.90 m. The walls of the clearstory present large splayed windows and oval openings that give access to the garret of the aisles. Also the walls that close the aisles present seven coupled column-buttresses systems reaching the height of 6.87 m and further splayed windows.

During the centuries, the complex suffered some esthetical modifications: the main prospect was modified since the narthex was eliminated installing an elaborate portal with a large rose-window; a part of the roof and of the lantern were rebuilt, introducing a Baroque style skylight turret; additional modifications on the roofing of the church were applied, with the reduction of the slope of pitches and with the restoration of the same slope as in the original form.

In order to determine the actual geometry and the mechanical features of the main constructional elements, an accurate experimental activity has been developed. In particular, both in situ inspection and laboratory tests have been carried out [6, 7].

It has been determined that the basic material constituting the constructional elements of the church is a very compact sedimentary limestone. In particular, columns and buttresses are made of plain stones with fine mortar joints (thickness less than 1 cm). The lateral walls (total thickness 120 cm) consist of two outer skins of good coursed ashlar (the skins being 30 cm thick) with an internal cavity with random rubble and mortar mixture fill.

In order to inspect the hidden parts of the constituting structural elements, endoscope tests have been executed on the right and left columns of the first bay, on the third buttress of the right aisle, on the wall of the main prospect and at the end on the filling of the vault covering the fourth bay of the nave. The test on the columns (Fig. 3a) allowed the exploration of the internal nucleus of the pier, relieving a total lack of internal vacuum, with the predominant presence of limestone connected with continuum joints of mortar (Fig. 3b). The test on the buttress was performed at the level of 143 cm, reaching the centre of the internal wall. The presence of regular stone blocks having different dimensions and connected to each other with mortar joints without any significant vacuum was detected. The tests on the wall put into evidence the presence of a two skins and rubble fill. The test made on the extrados of the vault, with a drilling depth of 100 cm, allowed a first layer of 7 cm made of light concrete and then a filling layer of irregular stones and mortar with the average thickness of 10 cm to be identified. In order to define the mechanical features of the material, original blocks of stone were taken from the cathedral and submitted to compression tests (Fig. 4a). In total, 10 different specimens having different sizes have been tested, giving rise to an average ultimate strength of about 140 MPa and an average density $\gamma = 1700$ kg/m^3. Besides, based on the results obtained for three different specimens, a Young's modulus equal to 42.600 MPa has been assessed, while a Poisson's ratio equal to 0.35 has been estimated.

(a)  *(b)* 

Figure 3: Endoscope tests.

Also, mortar specimens were extracted from the first column placed on the left of the first bay, from the wall of the aisle on the right and from the wall on the northern side of the transept. The specimens were catalogued as belonging to either the external joints (external mortar) or to the filling material (internal mortar). Compression tests have been carried out according to the Italian provisions (UNI EN 1926:2001), relieving a noticeable reduction of the average compressive strength for the specimens belonging to the external mortar (3.33 MPa) with respect to the internal ones (10.30 MPa). Besides, the Young's modulus has been determined on three different mortar specimens, according to the UNI EN 1015-11:2001 provisions, providing values ranging from 8.33 MPa to 12.16 MPa.

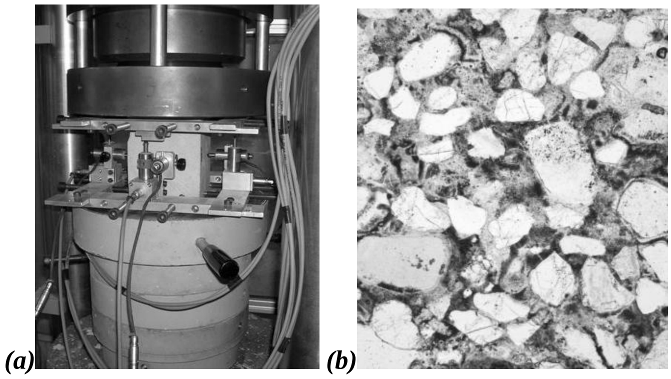


Figure 4: Compression tests on limestone (a) and microscope analysis on mortar (b).

Chemical and petrography analyses have been also performed on the mortar specimens. In particular, chemical tests were made by X rays diffractometer analysis, according to the UNI 11088:2003 provisions. The prevalence of three material, namely, quartz crystal SiO2, crystallized calcium carbonate CACO3 and some traces of felspate, was noticed. Also, a petrography study on thin sections of mortar specimens have been done by using two electronic microscopes, according to the UNI EN 932-3:1998 provisions (Fig. 4b). The analysis relieved the presence of quartz crystal sand end felspate, without any significant presence of crystallized calcium carbonate. The binding was quantified with a percentage of 60% of the total volume.

A FE model of the entire Abbey was calibrated on the basis of the in-situ experimental activity. The seismic analyses on such a model revealed that the more important structural part of the structural complex was to be recognized in the three-central bays of the main nave shown in Figure 5 [8]. For the above reason a physical model of the key-structural part was designed and constructed in the IZIIS Laboratory in Skopje (Fig. 6). The model was executed in a 1:5.5 scale ratio (length) which was the maximum value compatible with the capacity of the shaking table.

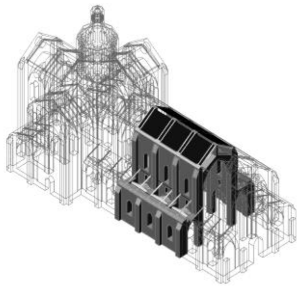
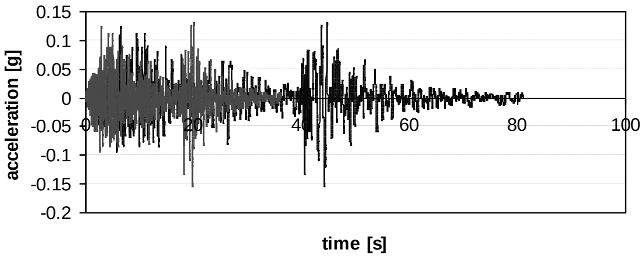


Figure 5. Recognized seismic resistant unit in transversal direction.

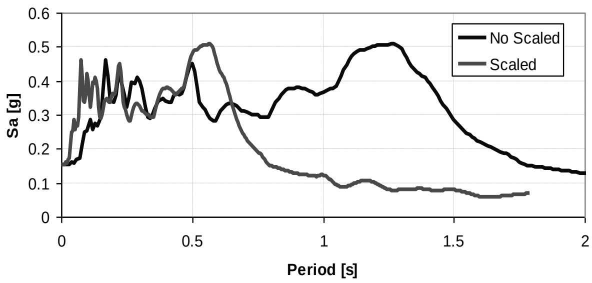
The Buckingham's theorem was followed to define all the physical parameters needed to the construction of the model, according to the "true replica" modeling principles. All the involved quantities was scaled on the base of the three main parameters Length (L_r=1/5.5), Mass Density (ρ_r=1) and Acceleration (a_r=1) so that for the stresses a scaling ratio σ_r=0.18 is obtained. The dimensions of the model were 3.97x4.44 m at the base, 3.67 m was the maximum height.

Fig. 6. Fossanova physical model (UPM) in scale 1:5.5 tested on shaking table.

Some simplifications were adopted in the construction of the prototype: free edges was left in longitudinal direction, in fact no boundary restrains was applied on the fronts, neglecting the longitudinal continuity as it is in the reality. Then, the wooden roof structure wasn't realized because it wasn't considered as an active element in the evaluation of the seismic vulnerability [9].



a. Calitri North-South record (scaled and not-scaled)



b. Relevant elastic response spectra

Figure 7. The accelerometric record (Irpinia, 1980).

The input signal of the test was assumed to be the scaled natural Calitri record (North-South direction) of Irpinia (Italy) 1980 earthquake record. The main feature of the selected earthquake are a maximum acceleration of 0.155g (compatible with seismic hazard of the site), a quite long duration time (80 sec), a high input energy for the relevant frequency (0.5Hz-10Hz) and typical two peak accelerations (or two strong motions). The record and the derived elastic spectra (with damping ratio ζ=5%) are shown in Figure 7a,b. The shaking table test was performed by considering three phases: phase 1, phase 2A and phase 2B. In the first phase the as-built unreinforced physical model (UPM) was tested and heavy damaged at the end. In the second phase (2A) the model was repaired and reinforced with carbon fiber ties. Finally, in the phase 2B, the reinforcing system was modified and the model was loaded until failure. Even though the examination of the reinforced systems is not the object of the present paper, in the following the level of input intensity which provoked serious damage to the model for every phase is listed:

- 0.14g for the original model (phase 1);
- 0.28g for the strengthened model (phase 2A);
- 0.40g for the strengthened model (phase 2B);

In particular, the maximum acceleration measured at the base of the shaking table, by means of the accelerometer "CH1" [11], versus the maximum absolute displacement measured at the top of the buttresses at each step of the phase 1 (unreinforced physical model) is shown in Figure 8. The curve can be assumed as an equivalent capacity curve for the tested UPM [10].

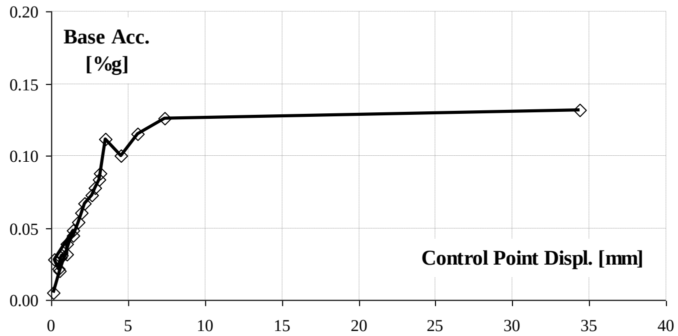


Figure 8. Equivalent capacity curve of the tested model.

3 DYNAMIC DAMAGE IDENTIFICATION

A more suitable representation of the structural response may be achieved by means of the decomposition in fully coherent independent vectors. Although the non gaussianity of the structural response vector $\mathbf{X}(t)=[X_i(t)] \quad (i=1,\ldots,n)$, collecting the nodal response processes, its main characteristics can be represented by the knowledge of the second order spectral properties. To this end, let us consider the PSD matrix of $\mathbf{X}(t)$

$$\mathbf{S}_X(\omega)=\begin{bmatrix} S_{X_1}(\omega) & S_{X_1X_2}(\omega) & \cdots & S_{X_1X_n}(\omega) \\ S_{X_2X_1}(\omega) & S_{X_2}(\omega) & \cdots & S_{X_2X_n}(\omega) \\ \vdots & & \ddots & \vdots \\ S_{X_1X_n}(\omega) & \cdots & \cdots & S_{X_n}(\omega) \end{bmatrix} \tag{1}$$

The elements of $\mathbf{S}_X(\omega)$ are the direct and cross power spectral densities, defined as the Fourier transform of the correlation components

$$S_{X_1X_j}(\omega)=\frac{1}{2\pi}\int_{-\infty}^{+\infty} R_{X_1X_j}(\tau)e^{-j\omega\tau}d\tau \tag{2}$$

or equivalently, starting from measurements of the process $\mathbf{X}(t)$:

$$S_{X_1X_j}(\omega)=\frac{1}{2\pi}\lim_{T\to\infty}\frac{E\left[X_i(\omega,T)X_j^*(\omega,T)\right]}{T} \tag{3}$$

where $X_i(\omega,T)$ denotes the Fourier transform of $X_i(t)$ over the observation time T

$$X_i(\omega, T) = \int_0^T X_i(t) e^{-j\omega t} dt \tag{4}$$

The psd matrix is Hermitian and non-negative definite, thus its eigenvalues $\mathbf{\Gamma}(\omega) = diag(\gamma_1(\omega)\, \gamma_2(\omega) \cdots \gamma_n(\omega))$ are real and non-negative, with orthonormal complex eigenvectors $\mathbf{\Psi}(\omega) = [\boldsymbol{\psi}_1(\omega)\, \boldsymbol{\psi}_2(\omega) \cdots \boldsymbol{\psi}_n(\omega)]$

$$\Psi(\omega)^{*T}\, \Psi(\omega) = \mathbf{I}, \quad \Psi(\omega)^{*T}\, \mathbf{S}_X(\omega) \Psi(\omega) = \Gamma(\omega) \tag{5}$$

$$\mathbf{S}_X(\omega) \Psi(\omega) = \Psi(\omega) \Gamma(\omega) \tag{6}$$

If the eigenvalues are sorted in decreasing order, then the summation in Eq.(9) can be truncated considering only a limited number of principal components.

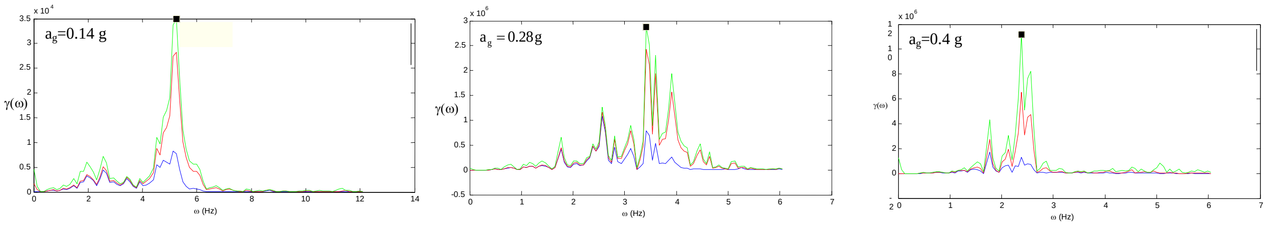


Fig.9 Response spectral eigenvalues for different values of ground peak acceleration:a_g=0.14g, a_g =0.28g and a_g =0.4g

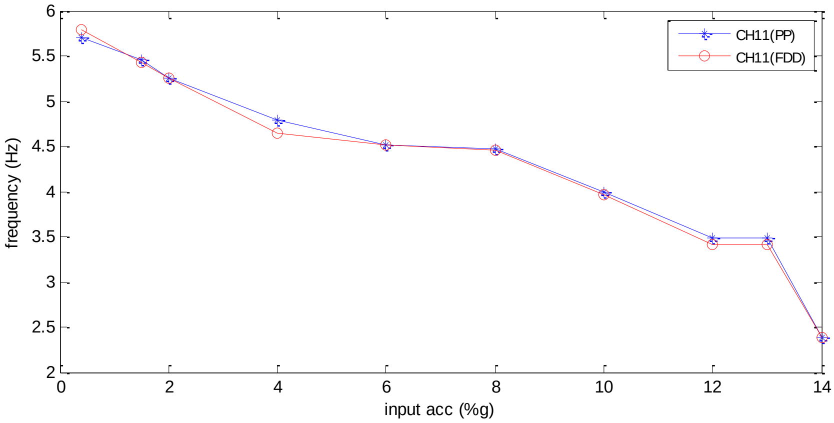


Fig.10 Comparison between experimental and identified frequency decay for different values of ground peak acceleration

Figures 9 show the response spectral eigenvalues for different values of ground peak acceleration, leading to the identification of the frequency decay, Fig. 10

Damage localization has been performed using the Parameter Method (PM) proposed by Dong et al. [14], [15] using a combination of frequency and mode shapes. The expression for the PM method is

$$\Delta\varphi = \sum_{j=1}^{n}\left[\phi_j^d\left(\frac{\omega_j^u}{\omega_j^d}\right) - \phi_j^u\right] \tag{10}$$

where ϕ is the structural mode, n the mode number while upper script u,d stands for undamaged and damage state respectively.

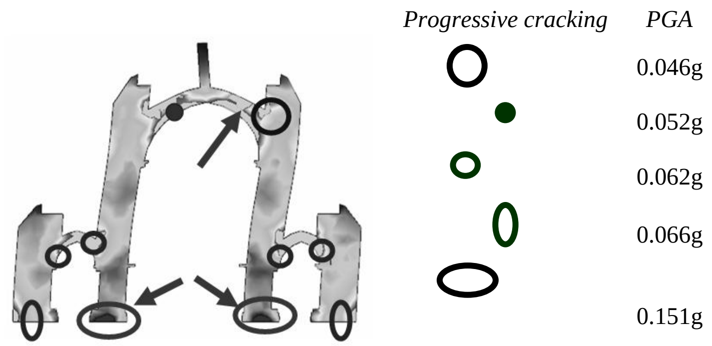


Figure 11. Progressive damage of the model with increasing PGA.

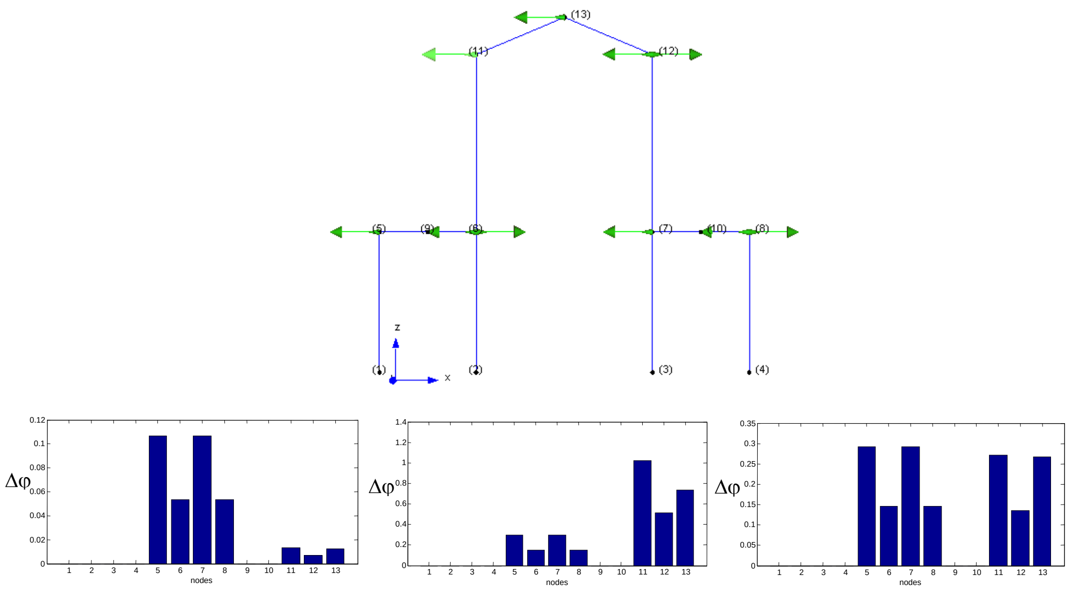


Figure 12. a) 2D finite element model [16], b) Progressive damage of the model with increasing PGA - PM method identification

Figure 11 show the FEM prediction of the progressive damage of the model with increasing PGA. In Figures 12, based on a 2D finite element model, it is shown the identification of damage location by the PM method.

CONCLUSIONS

The seismic behavior of a physical 1:5.5 scaled model of the church of the Fossanova Abbey has been investigated by means of numerical and experimental analyses. The achieved experimental results lead to the definition of a refined FE model reproducing the dynamic behavior of the whole structural complex. Then, the central transversal three-central bays of the church, as it mostly influences the seismic vulnerability of the Abbey, was investigated in a more detail by means of a shaking table test on a 1:5.5 scaled physical model in the Laboratory of the Institute for Earthquake Engineering and Engineering Seismology in Skopje. In the present paper a brief review of the numerical activity related to the prediction of the shaking table test response of the model is first proposed. Then, the identification of frequency decay during collapse is performed through decomposition of the measured power spectral density matrix. Finally, the localization and evolution of damage in the structure is analyzed. The obtained results shown that a very good agreement is achieved between the experimental data and the predictive/interpretative numerical analyses.

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