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Seismic vulnerability evaluation of the Fossanova Gothic church

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ABSTRACT: This paper focuses on the seismic behaviour of the church of the Fossanova Abbey, which represents a magnificent example of a pre-Gothic Cistercian style monument. Aiming at investigating the seismic vulnerability of such a structural typology, experimental and numerical analyses were carried out. Firstly, detailed investigations were devoted to the identification of the geometry of the main constructional parts as well as of the mechanical features of the constituting materials. Then, both ambient vibration tests and numerical modal identification analyses by finite element method were applied, allowing the detection of the main dynamic features of the church. Finally, a refined FEM model reproducing the dynamic behaviour of the structural complex were developed. Based on such a model, a numerical study was carried out, which, together with a global collapse mechanism analysis, allowed the identification of the most vulnerable parts of the church, providing also an estimation of the actual seismic vulnerability.

1 INTRODUCTION

Gothic architecture spread from the 12th Century in 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. Monastic orders and in particular the Cistercian one, with its monasteries, had an important role to broaden the new architectonic message, adapting to the local traditions the technical and formal heritage received by the Gothic style (Grodecki 1976, Gimpel 1982, De Longhi 1958).

Gothic cathedrals may be particularly sensitive to earthquake loading. Therefore, within the European research project "Earthquake Protection of Historical Buildings by Reversible Mixed Technologies" (PROHITECH), this structural typology is going to be investigated by means shaking table tests on large scale models (Mazzolani 2006). 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), was selected as an interesting and reference example of pre-Gothic style church (De Matteis et al., 2007a).

In such a paper, based on a previous experimental investigation devoted to the identification of the geometry of the main constructional parts, as well as the mechanical features of the constituting materials of the cathedral (De Matteis et al., 2007b) , the seismic behaviour of the Fossanova church is investigated. To this purpose, Ambient Vibration Tests were also carried out (De Matteis et al., 2007c). Therefore a refined FEM model reproducing the dynamic behaviour of the church has been developed and shown in this paper, allowing, together with a global collapse mechanism analysis, the identification of the most vulnerable parts of the church and providing an estimation of the seismic vulnerability.

2 THE FOSSANOVA CHURCH

2.1 *Geometrical features*

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. (2007b).

Figure 1. The Fossanova church.

Figure 2. The vaulted system of 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 9.15×8.85 m.

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 7 couples of cross-shaped piers (with dimensions of 1.80×1.80 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 7 coupled column-buttresses systems reaching the height of 6.87 m and further splayed windows.

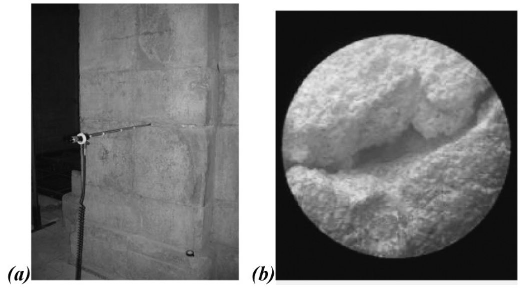


Figure 3. Endoscope tests.

During the centuries, the complex suffered some esthetic modifications: the main prospect was modified since the narthex was eliminated instead 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.

2.2 *Material identification*

To determine the actual geometry and the mechanical features of the main construction elements, both in situ inspection and laboratory tests were carried out. The basic material 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 were 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, revealing 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 height 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 two skins and rubble fill. The test made on the extrados of the vault, with a drilling depth of 100 cm, shown 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.

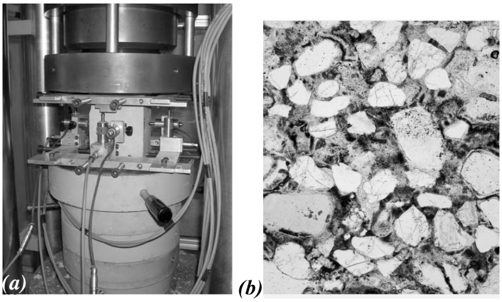


Figure 4. Compression tests on limestone (a) and microscope analysis on mortar (b).

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 of different sizes were tested, giving rise to an average ultimate strength of about 140 MPa and an average density $\gamma = 1700\,\text{kg/m}^3$. Besides, based on the results obtained for three different specimens, a Young's modulus equal to 42.600 MPa was assessed, while a Poisson's ratio equal to 0.35 was estimated.

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 were carried out according to relevant provisions (UNI EN 1926, 2001), revealing 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 was determined on three different mortar specimens, according to the European provisions UNI EN 1015-11 (2001), providing values ranging from 8.33 MPa to 12.16 MPa.

Chemical and petrography analyses were 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 were 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 as being the 60% of the total volume.

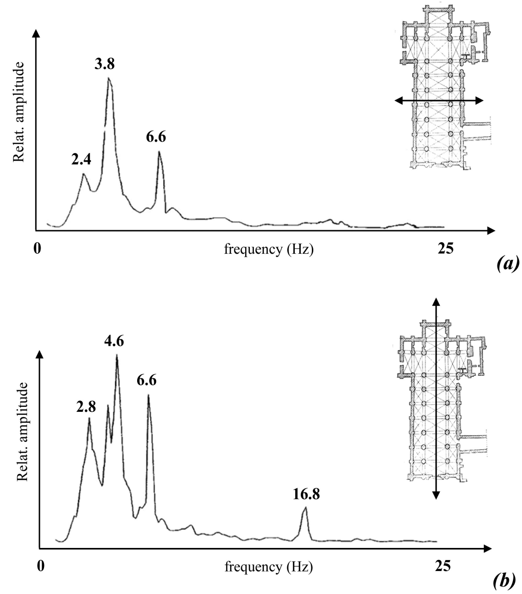


Figure 5. Fourier amplitude spectra: longitudinal (a) and transversal (b) direction.

2.3 *Dynamic identification*

The dynamic features of the whole cathedral were estimated by ambient vibration (e.g. human activity at or near the surface of the earth, wind, running water, etc.) tests, by measurements on several points of the façade, vaults, aisles and main nave. The test was performed in cooperation with the Institute of Earthquake Engineering in Skopje, by using 3 Ranger seismometers SS-1 (Kinemetrics production), 4 channels signal conditioner system and two-channels frequency analyzer Hewlet Packard for processing recorded time histories of ambient vibrations in frequency domain and to obtain Fourier amplitude spectra.

Detailed measurements were made both on the central frame and on the outer frames. The analyses were performed in two in-plane orthogonal directions, considering many pre-selected points along the columns and buttresses length. In addition, some measurements in vertical direction were carried out on the arches and vaults of the roof plan. The total number of measuring points was 25, of which 24 recorded on different points of the cathedral, while one point on the bell tower (De Matteis et al. 2007c).

The test results obtained from ambient vibration measurements clearly shown the fundamental frequencies of the cathedral in the main horizontal directions. As evidenced by the obtained Fourier amplitude functions depicted in Figure 5, along the transversal direction the first peak of the vibration response was at the frequency value $f = 3.8$ Hz (first transversal mode).

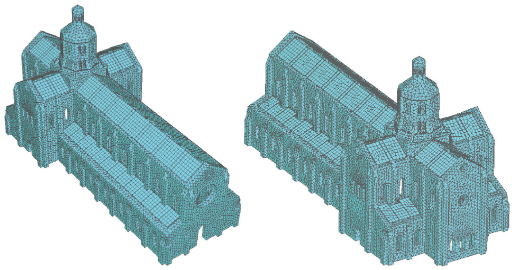


Figure 6. The developed FEM model.

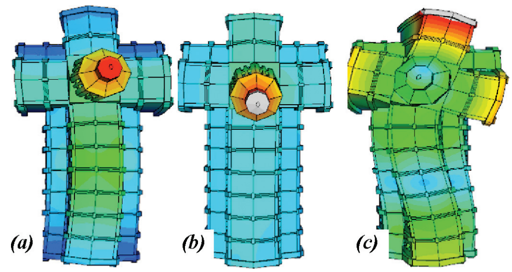


Figure 7. The obtained main vibrational modes: (a) transversal, (b) longitudinal and (c) torsional.

Instead, by the longitudinal direction the first peak of the response was at the value f = 4.6 Hz (first longitudinal mode). Both longitudinal and transversal tests indicated the first torsion mode at the value f ranging between 6.6 and 6.8 Hz. Moreover, it should be noted that the frequency values f in the range from 2.4 to 2.8 Hz were not taken into account as structural vibration; in fact, they were produced by external vibration (water flow or mechanical vibration of the nearby hydroelectric central).

3 THE NUMERICAL MODEL

3.1 *General*

Based on the results of the above experimental investigation, a detailed numerical FEM model of the whole cathedral (in full scale) was set up, completely reproducing the geometry of the church. For the model development, particular attention to the correct schematization of the main structural elements, namely walls, columns, vaults, buttresses and bell-tower, was paid (De Matteis et al. 2007d).

The FEM model, which was developed by using the software Abaqus (2006), is made by the assembling 425 different parts, namely 48 walls, 14 cross shaped columns, 50 shafts, 56 buttresses, 13 ogival arches on the nave, transept and apse, 32 Gothic arches on the aisle and on the chapels of the transept, 9 vaults covering the nave and the apse, 14 vaults on the aisles, 4 vaults on the transept, 4 vaults on the chapel of the transept and a vault on the corner between the aisle and the transept (each of them constituted by 8 groins), a bell-tower, 9 piers, the timber roofing structure (82 beam elements and 88 shell elements) (Fig. 6).

Additional inertial masses were located over the vaults of the nave, considering the presence of filling material made by stone and mortar mixture. An overall weight of 1.870 kg/m^2, distributed on each vault was assumed, through nine points of applications. Fixed joints at the base (ground level) of each element were hypothesized. In the whole, the adopted numerical model is constituted by 58.219 joints, with 217.571 solid elements and 4.976 shell elements, used for modeling the masonry elements. Besides, 4.759 joints were adopted to model the roofing structure (4.447 solid elements and 1.987 shell elements). The global mass of the entire model corresponds to about 22.100 t.

Table 1. Comparison in terms of detected natural frequencies.

	Detected natural frequency value [Hz]		
Direction	Ambient Vibration	FEM (1)*	FEM (2)**
Transversal	3.8	3.77	3.65
Longitudinal	4.6	4.63	4.30
Torsional	6.8	6.45	6.15

*with roofing wooden elements.
**without roofing wooden elements.

3.2 *Calibration of the model*

Elastic modal analyses were carried out to determine the natural frequencies corresponding to the main modal shapes. The adopted value of the Young modulus for masonry elements was initially E = 2.100 MPa, according to the Italian Seismic Code OPCM 3431 (2005). For the wooden roofing elements, a Young modulus E = 1.100 MPa was considered with a mass density $\rho = 450$ kg/m^3. Then, the structural model (model FEM 1) was calibrated against the experimentally measured frequencies, changing the estimated equivalent Young's modulus of the piers, vaults and walls, obtaining the values provided in Table 1. In Figure 7, the first three vibrational modes obtained by the applied numerical model are shown and the corresponding eigenvalues compared with the experimentally measured frequency values. It is apparent that a very good agreement is reached.

In order to evaluate the influence of the roofing structures, the above model was also developed without considering the roofing timber elements (model FEM 2). In such a case similar mode shapes were

obtained with the corresponding frequency values indicated in Table 1.

4 THE EVALUATION OF SEISMIC RESPONSE

4.1 *General*

The evaluation of the seismic response of the Fossanova church is carried out by means of both the finite element model analysis and the collapse mechanism analysis. In fact, by the finite element model the global response of the structural complex can be determined, allowing for the interaction among the different constituting parts. In addition, such an analysis allows the localization of the key parts of the church, i.e. the ones likely to incur into tensile cracking, where the location of the disconnections to be assumed in the rigid blocks mechanism analysis may be hypothesized. Then, once the conventional hinge locations was assessed, the mechanism analysis allows the determination of the seismic intensity (spectral pseudo-acceleration) producing the failure of the structural complex according to the predefined kinematic motion.

4.2 *FEM elastic analysis*

The dynamic response of the church is evaluated considering the Calitri record (Irpinia, 1980, Fig. 8a) and estimating the maximum effects by the elastic spectral analysis. To this purpose, the elastic response spectrum with a damping ratio $\zeta = 5\%$ was applied as input for the numerical analysis (Figure 8b). The main characteristic of the selected record can be identified in a quite long duration time (80 sec), a maximum acceleration (0.155 g) compatible with the seismic hazard of the site, a high input energy for the relevant frequency (0.5 Hz–10 Hz) and typical two peak accelerations (or two strong motions).

An elastic-linear behaviour was assumed for the constituting materials (masonry and wood). Basically, the numerical analysis is divided into the three following steps: (1) self-weight static analysis (2) modal analysis and (3) spectral analysis. It should be noted that the step (3) was carried out without considering the effect due to the self-weight. Therefore, the effects of step (1) was added to the minimum and maximum effects of step (3), according to typical combination rules.

The results achieved by the application of the model FEM (1) allows evaluating the influence of the roofing wooden elements on the local response of the arches and vaults. Basically, the roofing elements act as a sort of tie element for the masonry arches and vaults (Fig. 9), but the detected value of the maximum tensile stress acting on it, corresponding to 2.2 MPa, is not compatible with the existing simple connection

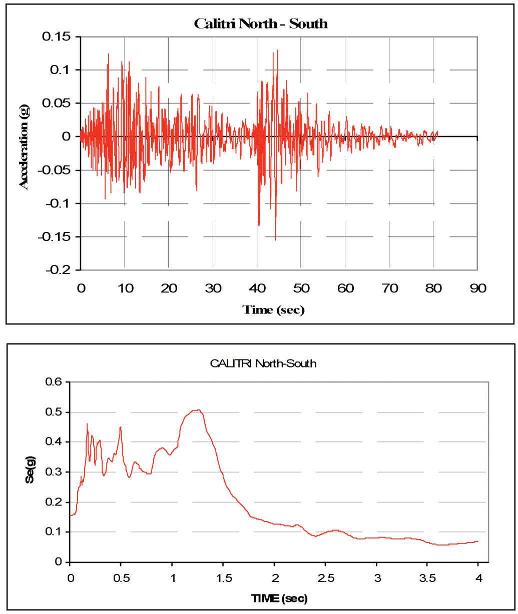


Figure 8. Calitri earthquake (North-South component): (a) acceleration (b) elastic spectra ($\zeta = 5\%$).

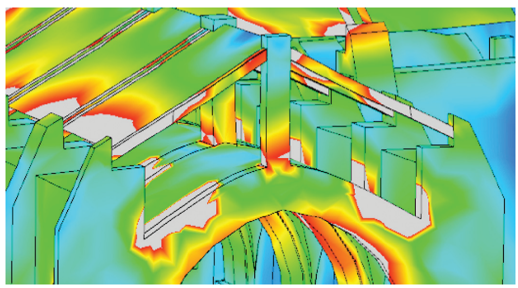


Figure 9. Effect of the roofing wooden element and evaluation of the tensile stress at the connection with masonry walls.

between such wooden elements and the supporting masonry walls. Therefore, avoiding complex nonlinear model of the material and masonry-wooden beam connections, an alternative numerical model FEM (2) was considered, where the presence of the wooden elements is neglected. Hence, assuming that the absence of the roofing wooden elements does not affect the evaluation of the seismic vulnerability of the church, the results of the numerical model FEM (2) are here considered.

The main outcome of the numerical model FEM (2) is shown in figure 10, where the deformed shape and the stress distribution related to the self-weight static analysis (Fig. 10a) and the dynamic spectral analysis

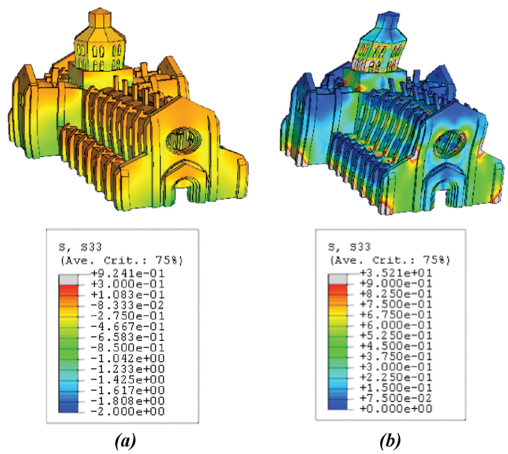


Figure 10. Deformed shape: Self weight static analysis (a) and dynamic spectral analysis (b) with related stress distribution.

(Figs 10b) are depicted. It is apparent that the vaults, the arches, the columns and the buttresses represent the portions of the structure where the peak stress values are attained.

For a deeper examination of the obtained results, in table 2, the stress values achieved in the key parts of the church are summarized (negative values are related to compression). It appears that the zones undergoing larger tensile stresses are the transversal arches (2.2 MPa) and the buttresses (1.3 MPa), while larger compressive stresses are attained at the base of the columns and buttresses, particularly in the transept and dome cladding areas where a peak value of −3.5 MPa is reached. Therefore, these zones should be considered as the key parts of the church, where collapse mechanisms are likely to occur.

Based on the results of the linear elastic analysis, it is possible to identify the parts of the church where cracking to tensile stresses are likely to occur, with the consequent disconnection of rigid blocks to create a kinematic chain.

In particular, concerning the nave arcade of the church, i.e. excluding the transept and dome cladding areas, if the conventional maximum material strength values for the masonry are assumed equal to 0.3 MPa and 5.0 MPa in tension and in compression, respectively, the possible locations of the hinges are the following (Fig. 11):

– two hinges in the central arches (level 20 m), where the tensile stress reaches 1.4 MPa;
– four hinges in the outer arches (level 8 m), where the tensile stress reaches 2.2 MPa;
– four hinges at the base of the buttresses and piers (level 0 m), where the tensile stress is 1.3 MPa.

Table 2. Stresses values in the key parts of the church.

Type of action	Stress value* [MPa]	Key part of the church
Gravity	−1.40	column base (dome cladding area)
Gravity	−0.95	column base (4th column of the central nave)
Gravity	−0.15	central nave arches (intrados side)
Gravity	−0.10	central nave arches (extrados side)
Gravity	−0.95	aisle arches (ext. side)
Gravity	−0.42	buttress of the 4th bay of the aisle
gravity + quake	1.20	central nave arches (intrados side)
gravity + quake	1.25	central nave arches (extrados side)
gravity + quake	1.80	bell tower
gravity + quake	2.20	aisle arches (ext. side)
gravity + quake	1.50	aisle arches (int. side)
gravity + quake	1.29	buttress of the 4th bay of the aisle
gravity + quake	0.33	column base (4th column of the central nave)
gravity + quake	1.50	transept buttress
gravity + quake	−3.50	transept buttress
gravity + quake	−2.00	bell tower
gravity + quake	−2.60	column base (dome cladding area)
gravity + quake	−2.30	column base (4th column of the central nave)

*Negative values stand for compression.

In order to follow step by step the progressive development of the zones incurring into tensile cracking, until reaching a kinematic motion, an incremental analysis was carried out as well and the earthquake intensity (peak ground accelerations) corresponding to the attainment of a local conventional collapse (a_c) were evaluated (see Fig. 11).

Based on such an analysis, for the applied peak acceleration $a_c = 0.046\,g$ the tensile stress reaches the conventional strength of 0.3 MPa in the main transversal arches of the nave arcade (level + 20,0 m) (Fig.11-label a). It should be noted that in such a position, the self weight static condition provides a stress value of −0.15 MPa. On the other hand, in the aisle transversal arches (level + 8.0 m) the conventional limit tensile strength is attained for $a_c = 0.062\,g$ (note that such arches are in compression for the self weight static condition). Also, in the buttresses the conventional

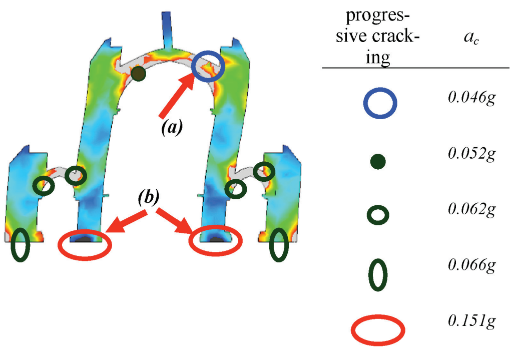


Figure 11. Location of the key parts subjected to larger stresses.

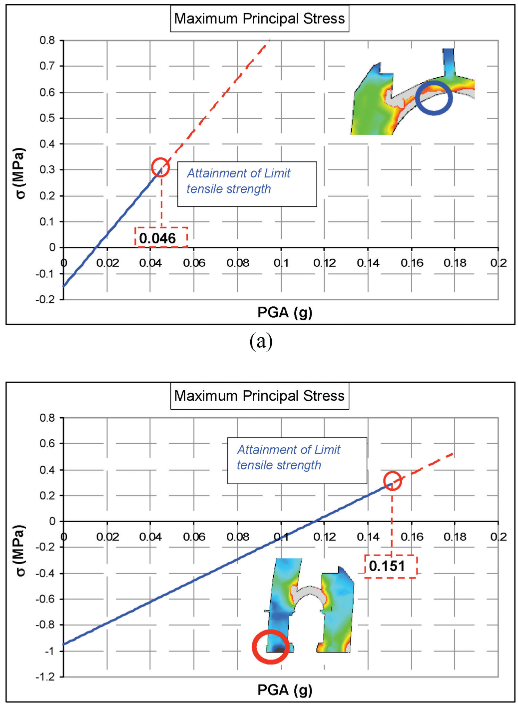
limit tensile stress is achieved for $a_c = 0.066\,g$. Eventually, for an applied acceleration $a_c = 0.151\,g$, all the columns undergo tensile cracking (Fig.11- label b). Therefore, it should be expected that a first damage limit state (DLS) condition should be attained for a PGA value $a_{c,DLS} = 0.046\,g$, while a complete collapse limit state condition (ULS) for a PGA value $a_{c,ULS} = 0.151\,g$.

For the sake of example, in figure 12, the variation of the stress with increasing PGA levels with respect to the initial stress condition (self weight static condition), up reaching the limit stress level producing the element disconnection, are provided for the transversal arch of the nave arcade (Fig. 12a) and the central piers (Fig. 12b), respectively.

4.3 *Collapse mechanism analysis*

In order to extend the previous elastic analysis, the results of a collapse mechanism analysis, which was carried out according to the provision of the Italian Seismic Code (2005), are provided. To estimate the actual seismic capacity of the structural complex, the main overturning local mechanisms of the church were analyzed (see Table 3), namely the ones concerning the prospects of the principal fronts (main façade, transept and apses), the lateral walls of the central body and the tympani of some prospects. Also, the analysis of the transversal arches of the main assemblages, considering the arches of the central nave and aisles, the arches delimiting the transept corps and the transept apses were carried out. Such elements have a peculiar function in the architectonic complex, with a predominate structural role respect to other macro elements.

For any analyzed mechanism, the collapse coefficient "λ", i.e. the multiplier factor of the horizontal equivalent forces producing the activation of the considered collapse mechanism, is firstly determined. Then, the corresponding spectral seismic acceleration a_0^* is evaluated for every local collapse mechanism



(a)

(b)

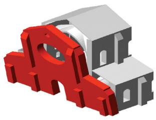
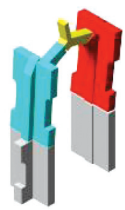
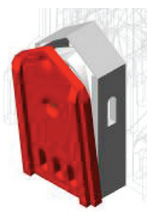
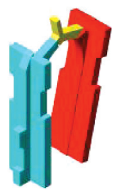
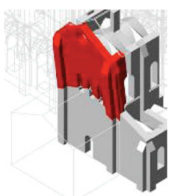
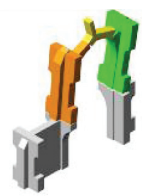
Figure 12. Stress variation with PGA level in the arch of the central nave (a) and at the basis of the central piers (b).

according to the codified procedure, which is based on the following main hypotheses: masonry has no tensile strength, sliding between the interconnected rigid blocks does not occur; 3) masonry has an unlimited compressive strength.

The obtained results are shown in Table 3, where the maximum spectral acceleration corresponding to the system collapse (kinematic motion of the rigid blocks) is provided for every hypothesized local mechanism. The lowest capacity is related to the overturning of the transept north prospect wall ($a_0^* = 0,06\,g$). On the other hand, a specific situation is related to the south prospect of the transept, where an adjacent construction is present, determining a rigid block mechanism of the only part of transept wall located at a level over $+12.25$ m, resulting in a reduction of the wall criticism.

As far as the local collapse mechanisms are concerned, the effectiveness of the transversal connection between the outer layers of the walls may have a strong influence on the capacity of simple elements against overturning collapse mechanism. In fact, when considering the cortical layers of the lateral walls disconnected from the inner part, a significant reduction of investigated capacity is noticed. For the sake of the example the collapse mechanisms of the aisle lateral

Table 3. Analyzed collapse mechanisms and corresponding capacities.

Case	Local collapse mechanisms	Spectral acceleration corresponding to the activation of the mechanism	Case	Local collapse mechanisms	Spectral acceleration corresponding to the activation of the mechanism
1)	*Overturning of the west prospect wall* 	$a_0^* = 0.084\,g$	*5)*	*Transversal arch (nave and aisles)* 	$a_0^* = 0.159\,g$
2)	*Overturning of the transept north prospect wall* 	$a_0^* = 0.058\,g$	*6)*	*Arch of the apse (type a)* 	$a_0^* = 0.215\,g$
3)	*Overturning of the apse east prospect wall* 	$a_0^* = 0.080\,g$	*7)*	*Arch of the apse (type b)* 	$a_0^* = 0.180\,g$
4)	*Overturning of the transept south prospect wall* 	$a_0^* = 0.132\,g$	*8)*	*Arch of north transept* 	$a_0^* = 0.213\,g$

wall and of the tympani of some prospects are provided in Table 4 and Table 5, respectively, considering both effective and not effective the transversal connection between the outer skins of the wall. In the latter case, the actual capacity of the considered collapse mechanism is strongly reduced. In particular, the vulnerability of the superior timpani of the principal church façade of the apses and of the transept is really remarkable.

As far as the arch systems are concerned, the minimum resistance was obtained for the nave arcade, which provided a spectral acceleration corresponding to the activation of the mechanism $a_0^* = 0.16\,g$. The considered mechanism is related to the development of seven bars-chain as shown in figure 13. The location of the conventional hinges was fixed by the outcomes of the linear elastic finite element analysis, based on the localization of the parts subjected to higher tensile stress concentrations.

4.4 *Vulnerability assessment*

In order to assess the seismic vulnerability of the church it is necessary to compare the above determined structural capacity with the relevant seismic demand (a_{se}). To this purpose, it has to be mentioned that the Fossanova abbey is located nearby the historical village of Priverno (LT) – Italy, which is included in a zone with a seismic hazard characterized by site peak ground acceleration on rock $a_g = 0.25\,g$. On the other hand, according to the Italian seismic hazard maps, the

Table 4. Overturning of the aisle lateral wall.

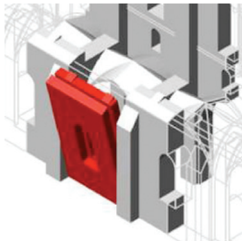
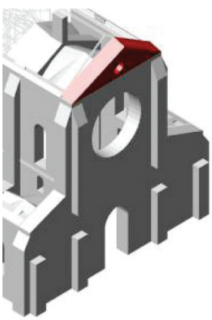
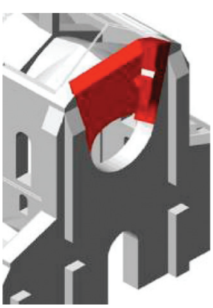
Spectral acceleration corresponding to the activation of the mechanism	
Effective transversal connection	*Not-effective transversal connection*
	
$a_0^* = 0.47\,\mathrm{g}$	$a_0^* = 0.17\,\mathrm{g}$

Table 5. Overturning of the west prospect tympanum.

Spectral acceleration corresponding to the activation of the mechanism	
Type a mechanism (horizontal disconnection)	
Effective transversal connection	*Not-effective transversal connection*
	
$a_0^* = 0.003\,g$	$a_0^* = 0.0005\,g$
Type b mechanism (vertical disconnection)	
Effective transversal connection	*Not-effective transversal connection*
	
$a_0^* = 0.004\,g$	$a_0^* = 0.0005\,g$

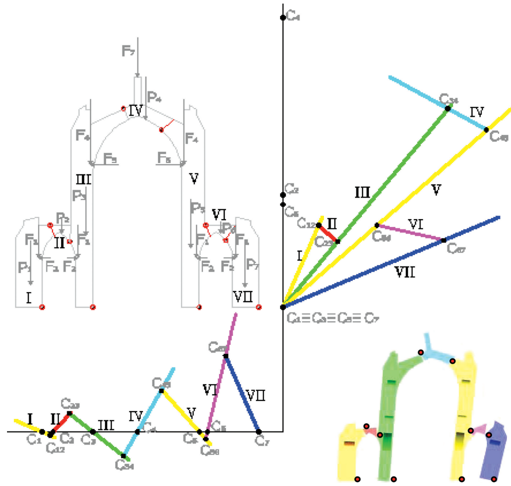


Figure 13. Location of the hinges for the collapse mechanism for the transversal arch (nave arcade).

estimated peak ground acceleration with an exceeding probability of 10% in 50 years is $0.125\,g$.

In order to evaluate the spectral accelerations (a_{se}) to be directly compared with the ones related to the activation of certain collapse mechanisms as determined in Section 4.3, according to the procedure suggested in the Italian Seismic Code (2005), the peak ground acceleration (a_g) has to be modified according to eq. (1):

$$a_{se} = \frac{a_g S}{q}\left(1 + 1.5\frac{H}{Z}\right) \tag{1}$$

where S is the sub-soil factor (here assumed equal to 1), q is the behavioral factor to account for the plastic energy dissipation capacity of the structure, which in the case being is fixed equal to 2, Z is the height of the centroid of seismic masses acting on the considered mechanism, H is depth of the structure. In Table 6, the main results are provided, where the spectral acceleration demands were determined according to both the above values of a_g, namely $0{,}25\,g$ and $0{,}125\,g$. In the same table, for the different analyzed mechanisms, the demand over capacity ratio a_{se}/a_0^* is also indicated. When considering a reference a_g value of $0.25\,g$, for all the examined cases the seismic demand is significantly larger than the structural capacity, meaning the activation of the selected collapse mechanism. Obviously, the situation improves when considering a reduced reference seismic acceleration ($a_g = 0.125\,g$). In particular, in such a case, the collapse mechanism no. 5, which is related to the transversal arch, is not activated, providing a η ratio lower than 1.

Table 6. Vulnerability assessment at ULS.

Collapse Mechanisms	Seismic demand (a_{se})		Demand-to-capacity ratio ($\eta = a_{se}/a_0^*$)	
Case according to Table 3	$a_g = 0.25\,g$	$a_g = 0.125\,g$	$a_g = 0.25\,g$	$a_g = 0.125\,g$
1)	0.19 g	0.10 g	2.38	1.25
2)	0.20 g	0.10 g	3.33	1.67
3)	0.21 g	0.11 g	2.63	1.39
4)	0.25 g	0.12 g	1.92	0.93
5)	0.21 g	0.11 g	1.32	0.69
6) FEM model (ULS)	0.25 g	0.125 g	1.66	0.83
7) FEM model (DLS)	0.125 g	0.062 g	2.70	1.35

The vulnerability assessment of the transversal arch may be also carried out according to the results derived by the FEM analysis. In such a case, the seismic demand has to be simply intended as the applied peak ground acceleration a_g, while the seismic capacity is measured according to the value of the acceleration producing the conventional collapse (a_c), which should be increased by a q-factor equal to 2 only when the conventional collapse is evaluated at the attainment of the first damage. The corresponding results are reported in Table 5, where the results related to FEM model (ULS) were obtained considering $a_c = 0{,}151\,g$ and $q = 1$, while the results related to FEM model (DLS) were obtained considering $a_c = 0{,}046\,g$ and $q = 2$. In the former case the obtained results are in a very good agreement with the ones related to the application of the local collapse mechanism analysis, even if with a significant increase of the demand over capacity ratio. On the contrary, in the latter case a significant deficiency of the transversal arch is evidenced as well as for an applied peak ground acceleration $a_g = 0.125\,g$.

5 CONCLUSIONS

In this paper, the seismic response of the church of the Fossanova Abbey was investigated. To this purpose, the dynamic features of the structural complex were identified by means of experimental (including ambient vibration tests) and numerical analyses. In particular, a refined FEM model reproducing the dynamic behaviour of the whole structure was developed. Based on such a model, a numerical study was carried out, which, together with a global collapse mechanism analysis, allowed the identification of the most vulnerable parts of the church, providing also an estimation of the actual seismic vulnerability.

In the whole the obtained results put into evidence that several parts of the church are significantly exposed to suffer damage, since they are unable to withstand the expected seismic demand. In particular, the most critical elements of the church are the north façade of the transept, which could collapse for the out-of-plane overturning, and the transversal arches of the nave, which could exhibit local failure mechanisms, when subjected to horizontal forces. Also, the important effect provided by transversal connection of the outer layers of the walls, whose actual effectiveness produces an important increase of the structural capacity against local collapse mechanisms in case of seismic event, was emphasized.

Preliminarily, it is important to observe that the analysis presented in this paper should be extended in order to contemplate in more detail the collapse of other elements, as for example the piers located in the dome cladding area and the bell tower. The seismic analysis of the nave arcade is particularly interesting since it involves many important structural elements, namely the transversal central arch, the aisle arches, the piers and the buttresses, which interact to each other, developing a complex failure mechanism. In addition, this part of the church represents a repetitive scheme, which is also present in many other historical buildings of the same epoch and characterized by a similar geometry. Therefore, such a macro-element may be assumed as a reference element characterizing such a kind of building typology (De Matteis et al. 2007a). The analyses developed in this paper, which are based on simplified and cautious assumptions, have shown that this macro-element is particularly exposed to significant damages for a seismic intensity lower than the one which is expected in the site where the Fossanova abbey is located.

A better estimation of the actual structural capacity under horizontal actions could be obtained only taking into account the actual interaction between such elements as well as considering the actual material features. Therefore, the dynamic response of the central part of the Fossanova church, including three consecutive bays, will be shortly investigated in a more accurate detail, it representing the prototype to be tested on a shaking table in a reduced scale (1-to-5.5) physical model, whose construction is now in progress at the IZIIS laboratory of Skopje (Republic of Macedonia) (Fig. 15).

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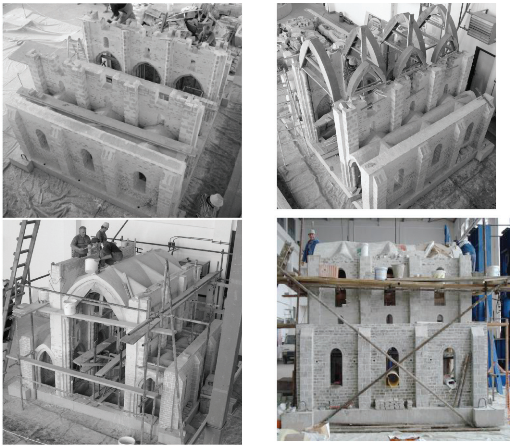

Figure 14. Prototype under construction (length scale 1:5.5).

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