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Seismic Vulnerability Assessment of the Casamari Gothic Church

G. de Felice[1], A. Genoese[1], A. Genoese[2] and M. Malena[1]
[1]Department of Engineering, Roma Tre University, Rome, Italy
[2]Department of Architecture, Roma Tre University, Rome, Italy

Abstract

This paper focuses on the seismic behaviour of the Casamari Gothic church located close to Frosinone in Italy. Linear static and dynamic finite element analyses are developed using a three-dimensional model of the whole church employing the commercial code ABAQUS. The three-dimensional model was realized using shell elements to model the midplane masonry walls. Fundamental frequencies of the cathedral in the main horizontal directions are derived by means of the finite element analysis.

Non-linear static analysis on a transversal section with horizontal forces adopted to represent the seismic action, known as pushover analysis, has been carried out to estimate the seismic response of the structure. The analyses have been carried out under conditions of constant gravity loads and monotonically increasing horizontal loads. In this case, the finite element model will be a two-dimensional plane stress model. In particular, masonry has been schematised as an anisotropic medium, whose yield criterion is derived using homogenisation, starting form a block structure with periodic texture. The adopted macroscopic model is shown to retain memory of the mechanical characteristics of the joints and of the shape of the blocks.

Keywords: masonry churches, structural models, modal finite element analysis, non-linear finite element analysis, homogenization, seismic capacity, ABAQUS.

1 Introduction

As demonstrated by several seismic events, such as the Emilia (2012) and the L'Aquila earthquakes (2009), historical churches are very vulnerable to earthquake motions. In fact, designed to withstand vertical loads, churches in general present slender walls, lack of horizontal structures, weak or nonexistent connections among structural elements, absence of effective tie-rods to absorb arch thrusts, and irregular stone texture.

These aspects, added to poor material performance especially in tensile stress mode, are among the reasons for the structural collapses of historical monumental buildings when subjected to seismic forces.

Approaches usually aim at evaluating the load value that implies the collapse of the each macroelement in which the church is divided, while numerical analyses of the whole building are generally unemployed because the collapse of masonry churches is rarely global. Recent works (see [1], [2] for instance) however have demonstrated the importance of such analyses, even if limited to the linear range, in order to consider the interaction between different elements, to have a-priori information about the most vulnerable parts of the church, to understand the dynamic behavior of the building. This work will focus on the seismic behaviour of the Casamari Gothic church located near Frosinone in Italy. FEM analyses of the church will be developed employing the commercial code ABAQUS. A linear static and dynamic analysis will be carried out on a 3D Finite Element model of the whole church; the 3D model will be realized using shell elements to model the midplane masonry walls.

Non-linear static analysis on the transversal section with horizontal forces adopted to represent the seismic action, known as pushover analysis, will be carried out to estimate the seismic response of the structure. The analyses will be carried out under conditions of constant gravity loads and monotonically increasing horizontal loads: an uniform pattern based on lateral forces that are proportional to the mass regardless of their height distribution in the structure will be considered. In this case, the Finite Element model will be a 2D plane stress model. In particular, masonry will be schematised as an anisotropic medium, whose yield criterion is derived via homogenisation, starting form a block structure with periodic texture. The adoped macroscopic model is shown to retain memory of the mechanical characteristics of the joints and of the shape of the blocks.

2 Case study

2.1 Description of the structure

The present study aims at investicgating seismic behaviour of the Casamari Gothic Church, part of the Abbey of Casamari. The Abbey is situated in the territory of Veroli (Frosinone), on the Via Maria, mid-way between Frosinone and Sora (Lazio, Italy). The Abbey was built in 1203 and consecrated in 1217. It is one of the most important Italian monasteries of Cistercian Gothic architecture. The building is similar to that of the French monasteries, while the facade of the Church has an imposing portico outside. A wide double door arch allows the entrance into the monastery. Inside there is a garden whose central part is occupied by the cloister, from which, it's possible to enter the Church, which is a three-aisled basilical. Behind the altar stands the Abbey choir built in 1940. The plan layout with the longitudinal and transversal section of the Church are reported in Fig 1.

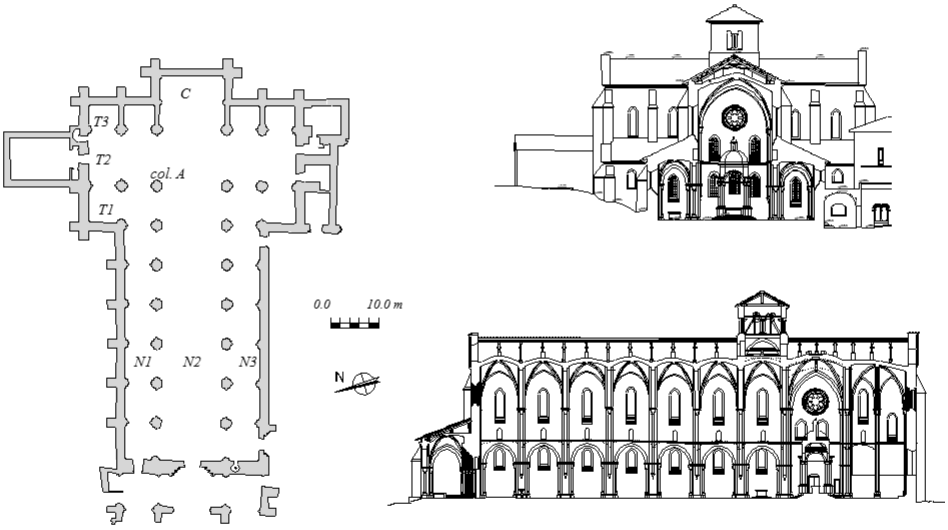

Figure 1: Plan layout, longitudinal and transversal view of the Church.

2.2 Geometry and materials

The main dimension of the central N_2 nave of the church is $61.3m$ with a width of 10.9 (Fig 1). The masonry wall thickness ranges between $1m$ (central nave walls) and $1.4m$ (lateral nave walls). The church roof is a timber structure. Columns, arches and vaults are single-leaf made of regular stone blocks with lime mortar joints, while solid bricks walls are inserted between timber roof and stone blocks walls. About mechanical characterization of the masonry, in the specific case, taking into account that the church does not show any particular or significant damage, a complete characterization of these material was not aimed. The masonry constitutive parameters have been derived from the Italian Guideline [3]. The adopted values are reported in Table 1.

Material	Young Modulus [MPa]	Poisson Coefficient	γ KN/m^3
Square stone Block Masonry	2.8E3	0.2	22.0
Solid brick masonry and lime mortar	1.5E3	0.2	18.0
Timber roof element	11E3	0.35	6.0

Table 1: Mechanical characteristics of masonry.

This values have been adjusted, by mean of suitable coefficients, to take into account the presence of thin joints between blocks, as suggested by the guideline.

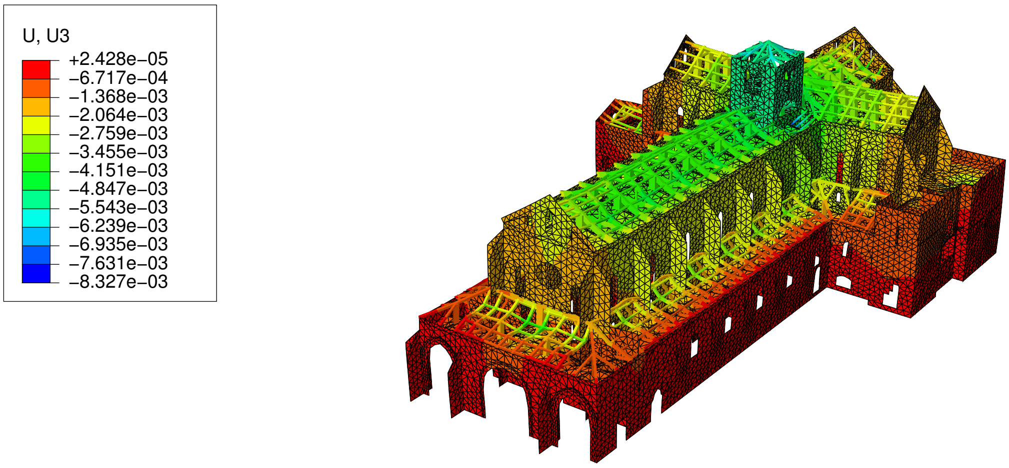

Figure 2: Vertical displacement under self-weight forces [m].

3 3D model

3.1 FE model and linear static and dynamic analyses

A 3D finite element model of the church has been realized using shell elements to model the midplane masonry walls and using truss and beam elements to model the timber roof structures of naves and aisles. Shell elements are also employed to model the vaults of the central N_2 and lateral naves N_1, N_3 (Fig 1), of the transept T_1, T_2 and T_3 and of the choir C (Fig. 1). The model consists of 55529 shell elements and 1785 truss or beam elements for a total of 34849 nodes. Boundary conditions have been applied restraing horizontal and vertical displacements for all the nodes at the ground level. A first linear analysis has been carried out on the 3D FE model using the code ABAQUS in order to find the solution under self-weight static actions. After, the main mode shapes and frequencies have been calculated.

Fig. 2 shows the vertical displacement under self-weight forces, while in Fig. 3 are reported the compressive stresses at the base of the column A, located under the bell tower (see Fig. 1). The latter are obtained on the basis of the Saint-Venant solution (see [8] for instance) once the resultants over the cross sections in terms both forces and moments have been calculated.

The results of the modal analysis are collected in Table 2, where the frequencies of the main mode shapes, together with the mass participation factor, are listed. Fig.s 4-6 show the first three selected eigen-modes. The first mode is a global eigen-mode in the trasversal direction, corresponding to a frequency equal to 3.0686Hz with a mass partecipation equal to 42.2% in transversal direction. The second mode is a global

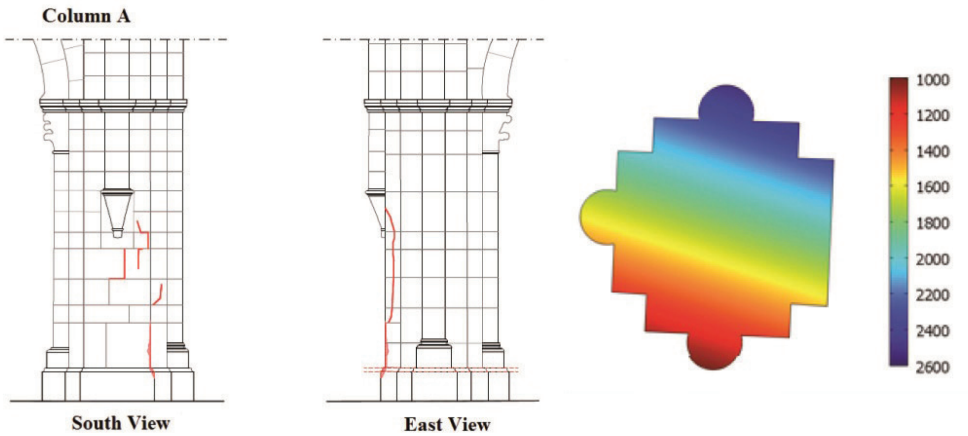

Figure 3: Compressive stresses [kPa] under self-weight forces.

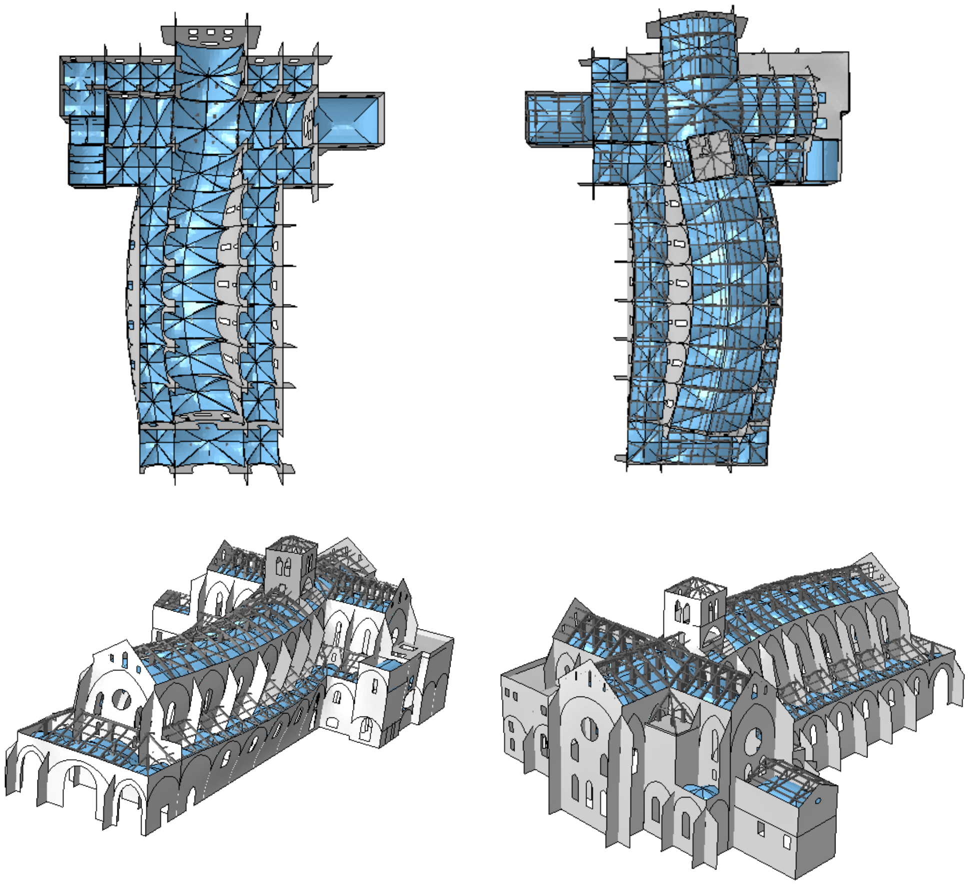

Figure 4: First eigen-mode: transversal mode.

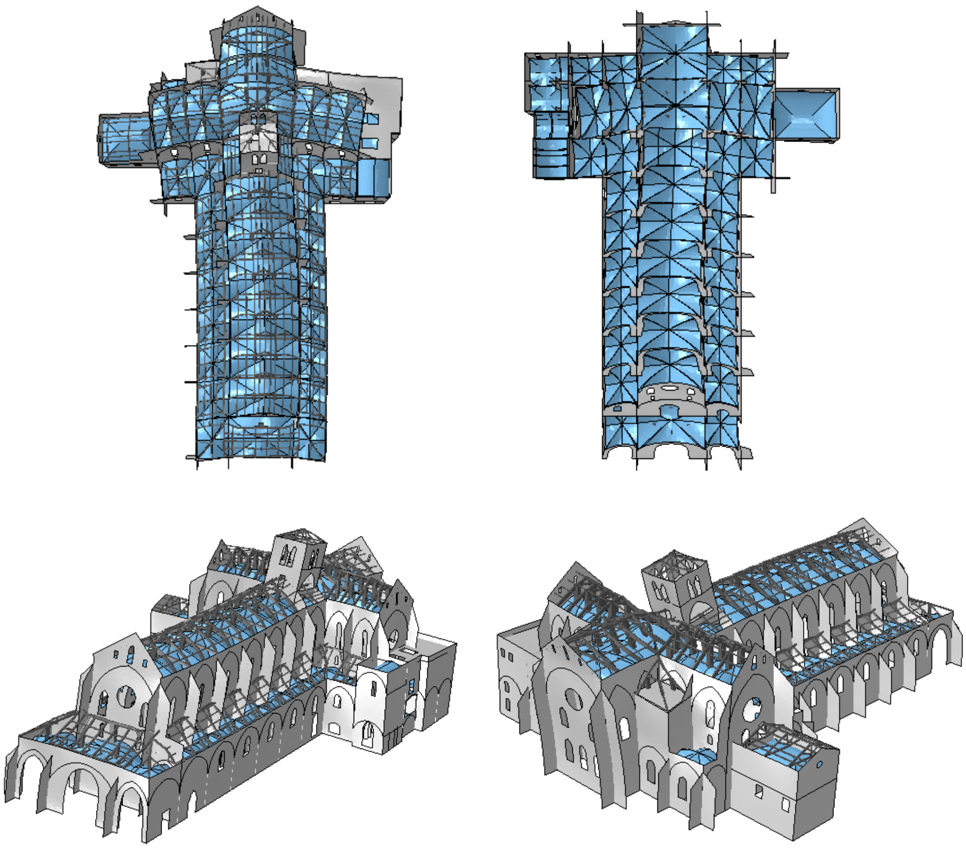

Figure 5: Second eigen-mode: longitudinal mode.

mode in longitudinal direction with a mass partecipation of 53.4% in longitudinal direction. The third mode is a global torsional mode with a mass partecipation equal to 11% and 1.72% in transversal and longitudinal direction, respectively. The other six eigen-modes, reported in the Table, are global or local mode, involving out of place mechanism of the facade. Mass participation sum of the first 9 eigen-modes results is equal to 62.7% and 55.4% of the total mass in the transversal and longitudinal directions, respectively.

It is interesting to note that the longitudinal mode shape, n. 2 (Fig. 7) involves the facade and the nave walls, while the transversal mode shape, n.1 (Fig. 7), involves the central and lateral naves with their vaults.

In the following sections, these modes are employed to suggest the failure mechanism of the building with the aim of selecting the principal macrolement to analyze by mean of a nonlinear static analysis in way to estimate the seismic response of the structure.

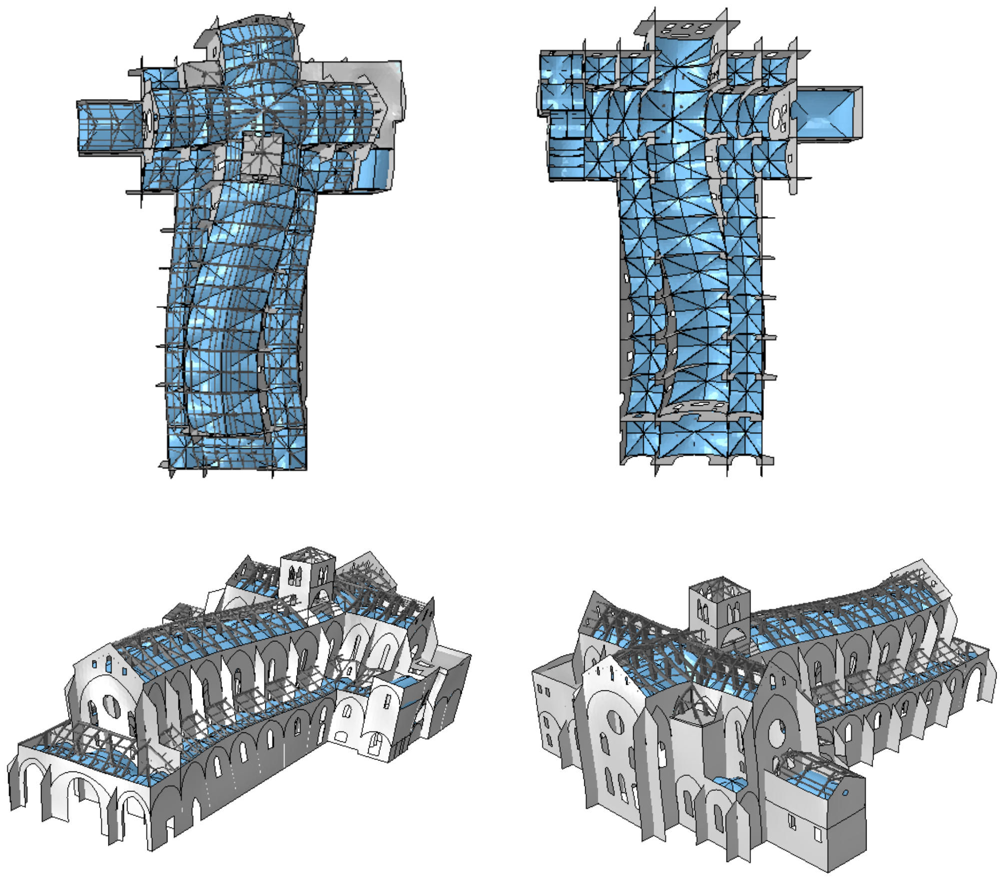

Figure 6: Third eigen-mode: torsional mode.

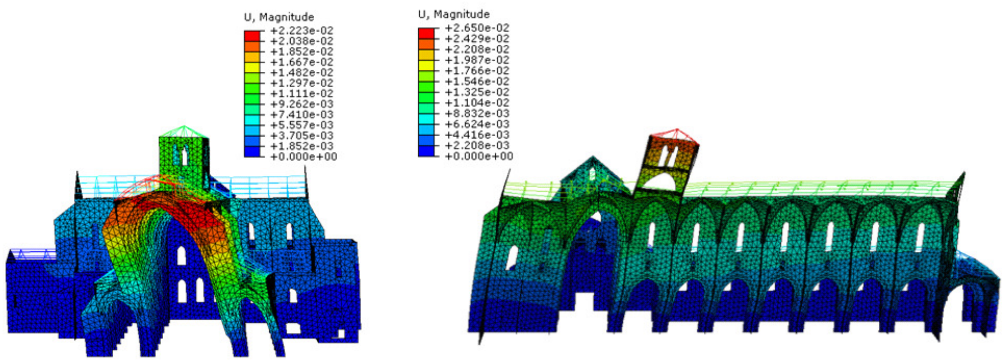

Figure 7: Transversal and longitudinal section of the church.

Mode n.	Frequency [Hz]	Mass participation Trasversal direction [%]	Mass participation Longitudinal direction [%]
1	3.0686	42.2	0.034
2	3.7751	0.018	53.41
3	3.9997	11.0	1.73
4	4.8769	0.053	0.0001
5	4.8800	0.136	0.018
6	4.8840	0.001	0.042
7	4.8847	0.005	0.084
8	4.9592	9.17	0.219
9	5.0636	24.3	0.0001

Table 2: Modes frequencies and partecipation factors.

4 Two-dimensional non linear analysis of macroelements

4.1 Constitutive model for masonry

The constitutive model for and masonry is formulated in the framework of classical rate-independent plasticity. A linear elastic-perfectly plastic constitutive model was selected for masonry where, however, anisotropy in both elastic properties and strength envelope are taken into account. The model is formulated in the framework of homogenization theory of periodic media, referring to a block masonry structure, consisting of a periodic pattern of elastic blocks with cohesive and frictional joints. In such a case, a closed-form approximated expression for the elastic strain energy is provided in [4] that takes the following form:

$$\mathcal{W}(\boldsymbol{\varepsilon}^e) = \frac{1}{2\left(1-\nu_{12}\nu_{21}\right)}\left(E_1\left(\varepsilon^e_{11}\right)^2 + E_2\left(\varepsilon^e_{22}\right)^2 + 2\nu_{12}E_2\,\varepsilon^e_{11}\varepsilon^e_{22} + 4G\left(\varepsilon^e_{12}\right)^2\right) \tag{1}$$

where x and z are the horizontal and vertical direction, respectively. The coefficients in (1) depend on the elastic Lamè coefficients of the blocks $(\lambda_b^{'}, \mu_b)$ and the normal K_n and tangential K_t stiffness of the joints, as well as on the height a and width b of the blocks as follows:

$$\frac{1}{E_1} = \frac{4a}{4abK_n + b^2K_t} + \frac{1}{4\mu_b} + \frac{1}{4\left(\lambda_b^{'} + \mu_b\right)} \tag{2}$$

$$\frac{1}{E_2} = \frac{1}{aK_n} + \frac{1}{4\mu_b} + \frac{1}{4\left(\lambda_b^{'} + \mu_b\right)} \tag{3}$$

$$\frac{1}{G} = \frac{1}{aK_t} + \frac{4a}{b^2K_n + 4abK_t} + \frac{1}{\mu_b} \tag{4}$$

$$\frac{\nu_{12}}{E_1} = \frac{\nu_{21}}{E_2} = \frac{\lambda_b^{'}}{4\mu_b\left(\lambda_b^{'} + \mu_b\right)} \tag{5}$$

In the context of multi-surface perfect plasticity, the elastic domain is defined as:

$$E_\sigma = \{\, \boldsymbol{\sigma} \mid f^\alpha(\boldsymbol{\sigma}) := \mathbf{n}^\alpha : \boldsymbol{\sigma} - c^\alpha \leq 0 \quad \forall\, \alpha \in [1,..,m]\} \tag{6}$$

where $f^\alpha(\boldsymbol{\sigma}) = 0$ are m independent planes, intersecting in a non smooth way, which define the yield surface. The expression for the yield surface of block masonry is provided in [5] on the basis of the yield design homogenization method. In particular, if the blocks are assumed as infinitely resistant bodies and the joints as frictional interfaces, with friction angle ϕ, the yield surface comprises $m = 4$ planes which can be written in terms of the stress components in the Ox_1x_2 reference adapted to the joints as follows:

$$\begin{array}{r}
f^1 := \mu\sigma_{11} + tg(\phi)\sigma_{22} + (1 + tg(\phi)\mu)\,\sigma_{12} \leq 0 \\
f^2 := \mu\sigma_{11} + tg(\phi)\sigma_{22} - (1 + tg(\phi)\mu)\,\sigma_{12} \leq 0 \\
f^3 := \sigma_{22} + 1/tg(\phi)\sigma_{12} \leq 0 \\
f^4 := \sigma_{22} - 1/tg(\phi)\sigma_{12} \leq 0
\end{array} \tag{7}$$

where x_1 and x_2 are the horizontal and vertical direction, respectively, while $\mu = 2a/b$ is the aspect ratio height-to-width of the blocks.

The model is integrated at the Gauss point level by means of a numerical procedure based on quadratic minimisation [6], as proposed in [7].

4.2 Non-linear static analysis

A transversal section af the church has been selected by mean of the results of the linear dynamic analysis.

The non-linear static analysis on this macroelement with horizontal forces adopted to represent the seismic action, has been carried out to estimate the main seismic response of the structure.

The mechanical properties of the constituent materials have been calculated starting from the base values of $3.6\times10^6 kPa$ and of $1.50\times10^6 kPa$ for the Young moduli of the blocks and the mortar joints respectively. These values are reduced by using a factor of 0.44 and 0.93 (i.e. the ratio between the effective and the geometric tickness of $6.15m$ considered for the planar model) for the regions denoted as transversal arches and lateral/central walls respectively (see Fig. 8) in order to account the openings. Coherently, the same reducctions are applyed also to the self-weight of the masonry and the vaults fill ($22kN/m^3$ and $18kN/m^3$ respectively) and to the base values of the cohesion calibrated for each part reported in Fig. 8. Finally, the mortar joints tickness and the friction angle are $1cm$ and 30 for all the regions, while different aspect ratios are introduced in order to consider the worst behaviour of the inner core of both the lateral walls core and the vauls fills (see Fig. 8).

The results of the push-over analysis are reported in Fig. 9, in terms of horizontal displacements, maximum plastic strains and load proportional factor vs horizontal displacement of the control point, highlighted in the same figure. The observed maximum

masonry element		effective thickness [m]	blocks aspect ratio μ [-]	joints cohesion [kPa]
lateral nave arches	(light blue)	0.60	0.600	170.0
central nave arch	(green)	0.60	0.600	80.0
lateral nave arches	(red)	1.00	0.600	170.0
central nave arch	(purple)	1.00	0.600	80.0
transversal arches	(cyan)	2.70	0.600	25.0
kurbs	(blue)	6.15	-	-
columns	(light grey)	0.60, 1.00, 1.50	0.600	25.0
central nave walls	(grey)	5.70	0.600	25.0
lateral naves walls	(salmon)	5.70	0.600	25.0
central nave walls core	(khaki)	2.70	1.714	25.0
lateral nave walls core	(dark purple)	2.70	1.714	25.0
vaults fill	(yellow)	1.00	1.714	10.0
buttress	(mint green)	0.95 - 1.90	0.600	25.0

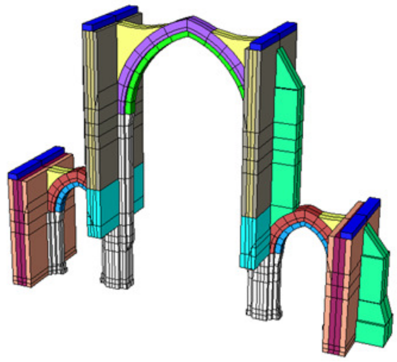

Figure 8: Transversal section of the church.

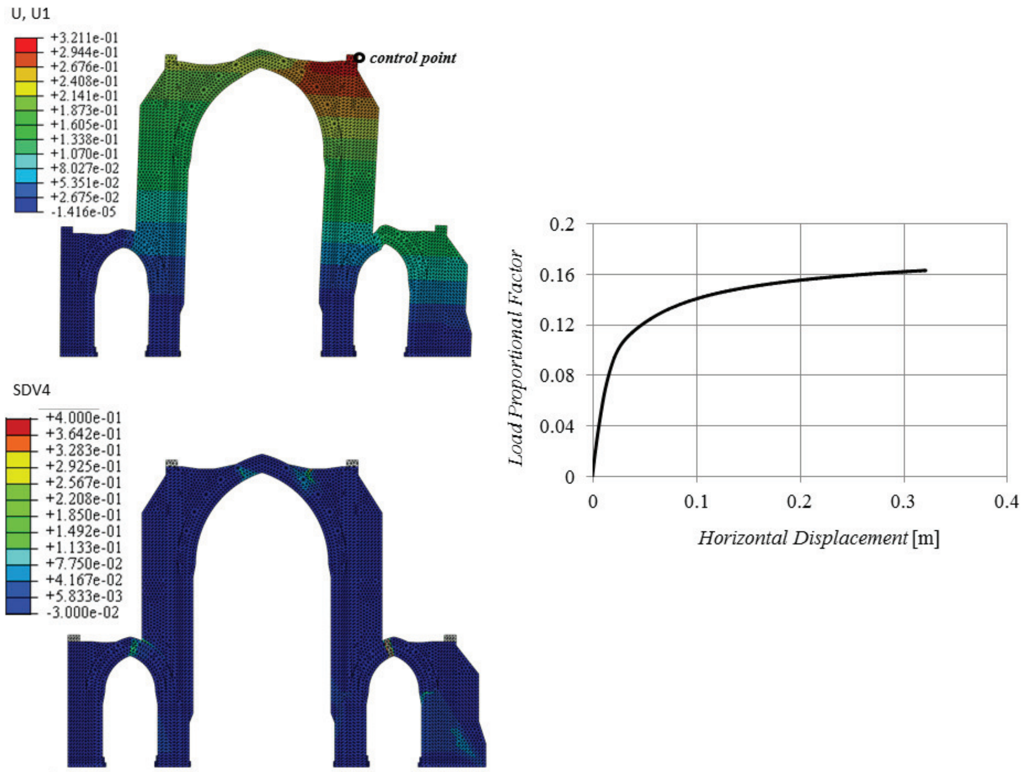

Figure 9: Push-over analysis results.

horizontal displacement is almost $30cm$, while the concentration of plastic strains, corresponding to the opening of the mortar joints, localizes mainly in correspondence of the central and of the two lateral arches.

5 Conclusion

A study of the seismic behaviour of the Casamari Gothic church has been presented. A linear static and dynamic finite element analysis has been developed on a 3D model of the whole church in order to find the solution under self-weight static actions and in order to calculate the main mode shapes and frequencies. These first analyses have given the possibility to study the interaction between the different structural elements and to have a-priori information about the most vulnerable parts of the church. By means of these informations, a transversal section of the church has been selected and a push-over of it has been carried out in order to study the seismic responce of the most vulnerable part of the structure.

Acknowledgements

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